CONCRETE
AND
CONSTRUCTIONAL ENGINEERING

INCLUDING PRESTRESSED CONCRETE

JANUARY 1960 — VOL. LV. NO. 1

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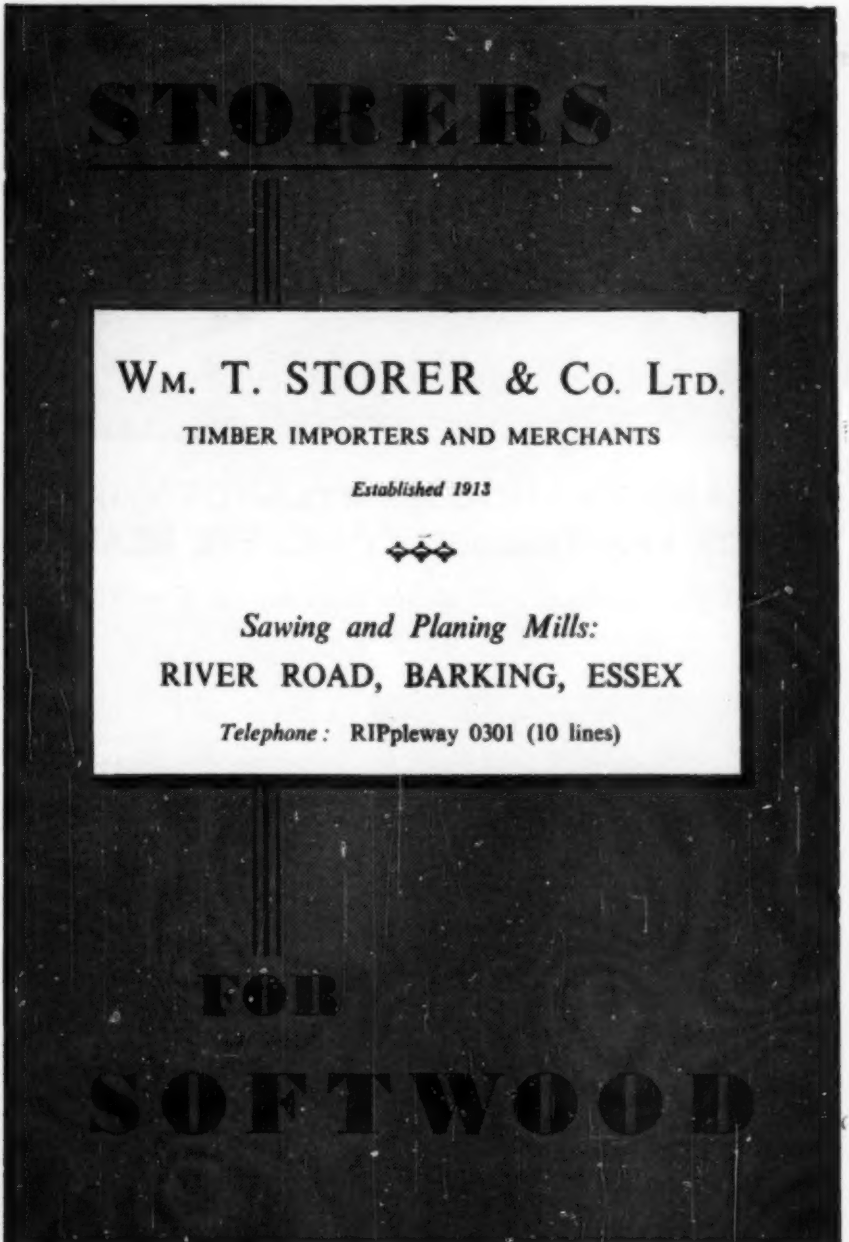
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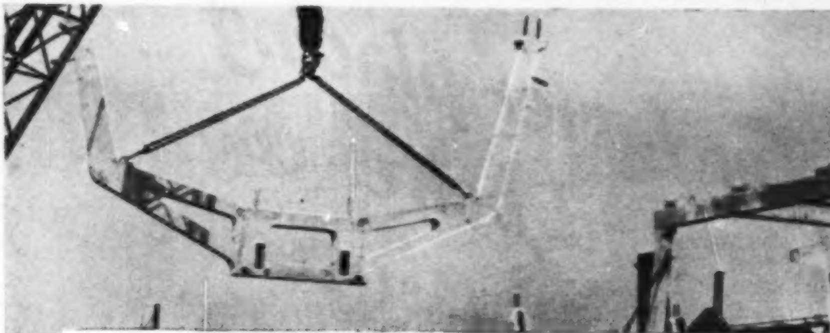

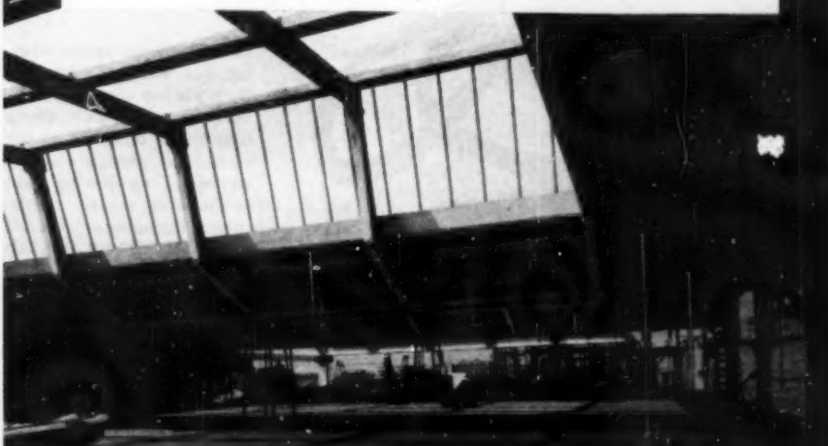


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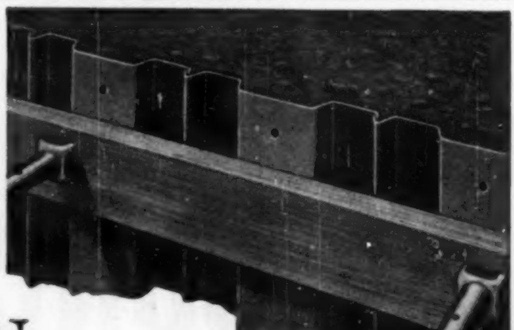

Designing and building handsome colliery structures like these is one of the ways in which Simon-Carves are helping to implement the N.C.B.'s current policy of retrenchment and modernisation.

The pictures were taken at Bilston Glen, where a new colliery is under construction for the Scottish Division of the N.C.B. Above: the men and materials winding headframe and airlock, and associated buildings, seen from the coal-winding headframe. Below: a general view of the site, with the washery and settling tank in the foreground. Simon-Carves Ltd. are responsible for all structural design at this new colliery, as well as for the design and supply of the skip and cage-winding plants and the coal preparation plants.

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A NEW LOOK FOR THE MINES

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Architectural design at Bilston Glen by E. Riss, Esq., F.R.I.B.A., F.R.I.A.S., Production Architect, and D. D. Jack, Esq., F.R.I.B.A., Welfare Architect.

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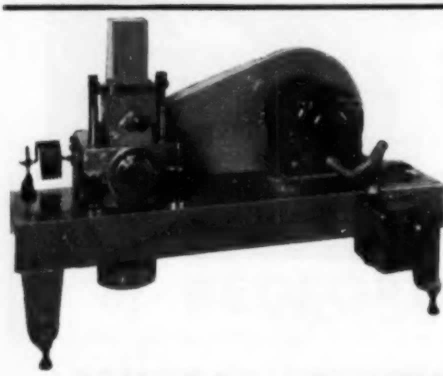

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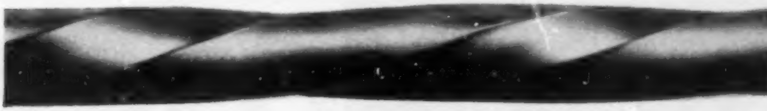

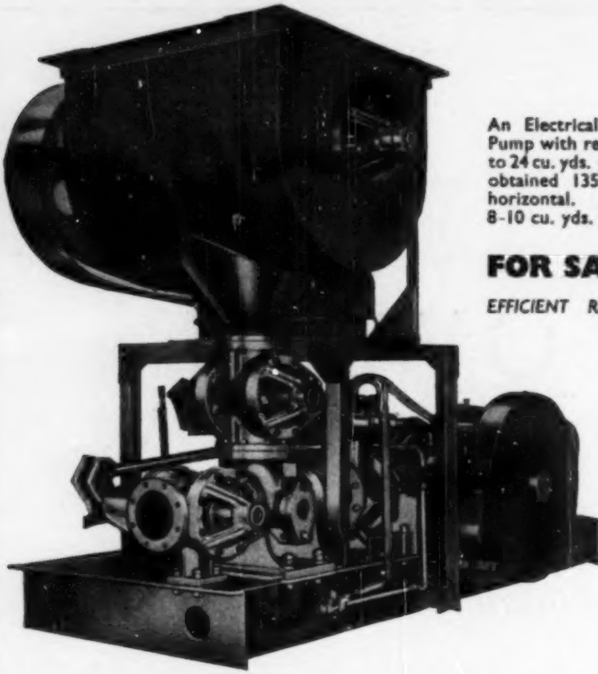

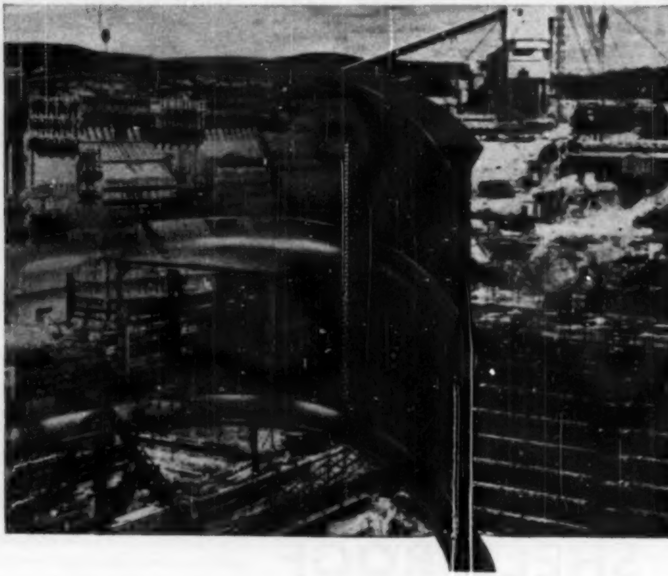

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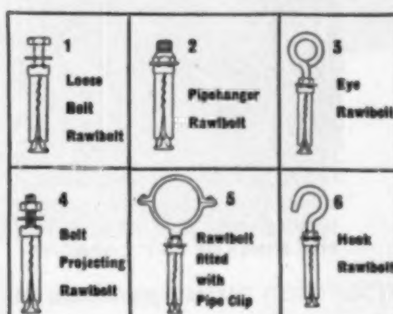

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It is impregnated by a new solvent injection process with a low volatile high residual binder guaranteeing saturation of all fibres.

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Resilex is not laminated or otherwise fastened together; is unaffected by road and runway grade of sealing compound and can be delivered in pre-selected sizes. Fully descriptive leaflet on request.

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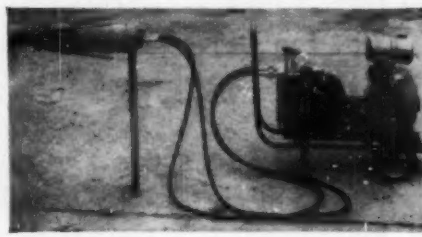
PSC for all

FREYSSINET SYSTEM

The illustration on the left shows the Freyssinet System of cone anchorage, with "Hydrarigid" sheathing. It is normally cast in the end of the member to be prestressed, and is usually recessed to permit a protective mortar covering to be applied after grouting. Anchorage cones are provided for 8/200, 10/200, 12/200, and 12/276 cables.

. . . CONE ANCHORAGE



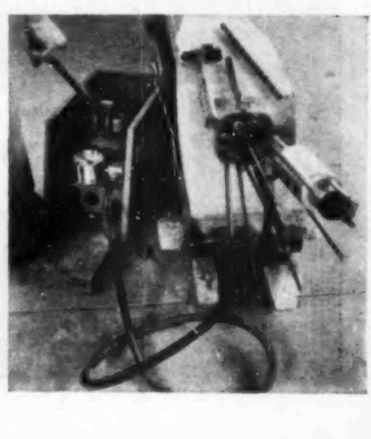
FREYSSINET POWER EQUIPMENT . . .

One of the most recent developments in the Freyssinet system of prestressing has been the introduction of power stressing. An electric or I.C. operated motor driving an hydraulic pump is trailer mounted and the hydraulic circuit has been simplified to only two controls which require the operator's attention. These are mounted on the trailer assembly and the jack is thus completely remote-controlled. The advent of power stressing has increased the rate of tensioning by three or four times that of hand pumping.

P.S.C. MONOWIRE SYSTEM

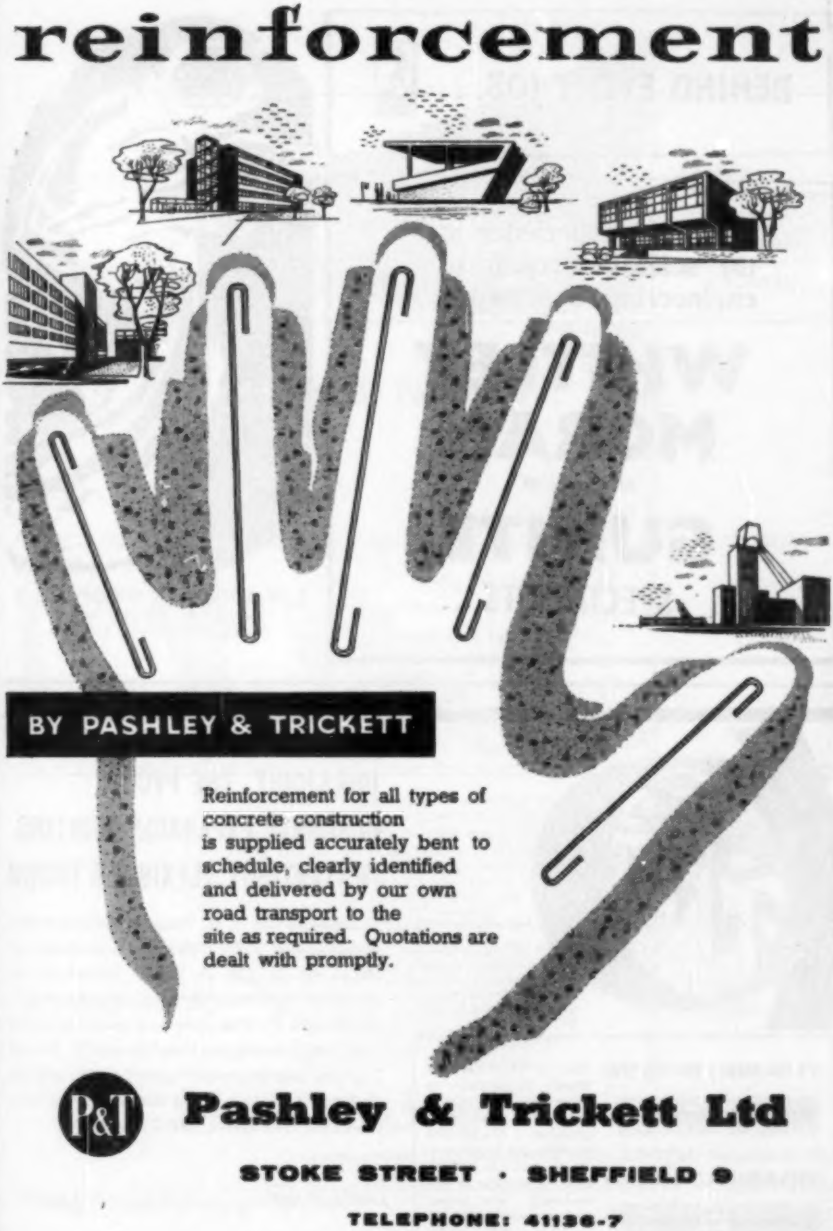
. . . P.S.C. ANCHORAGE

The 8/276 P.S.C. "MonoWire" Anchorage illustrated here shows a "Hydrarigid" welded seam-jointed metal sheath which screws into the anchorage. A new type of high-impact plastic cable spacer is also available. These anchorages can be supplied for cables of one to twelve wires.



P.S.C. MONOWIRE POST-TENSIONING . . .

This illustration shows the P.S.C. lightweight single-wire jack stressing one of twelve wires on a 12/276 "MonoWire" anchorage. The jack is available with 4" or 6" extensions, and has a spring return.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BY PASHLEY & TRICKETT
Reinforcement for all types of concrete construction is supplied accurately bent to schedule, clearly identified and delivered by our own road transport to the site as required. Quotations are dealt with promptly.
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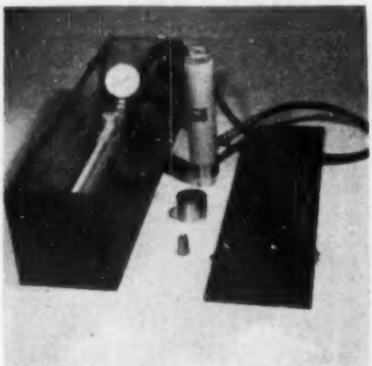
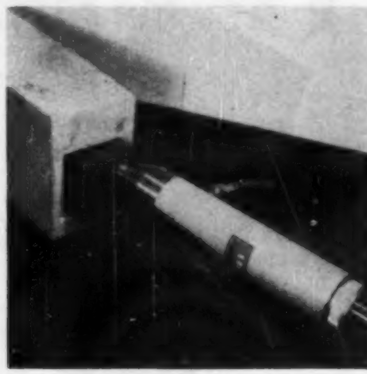
STRAND OR WIRE STRESSING

PRE- AND POST-TENSIONING

Here is an economical and portable stressing set of high quality. Interchangeable components enable the same jack to be used for stressing wire or strand.





Three standard forms are available:

1. For 3/8" and 7/16" diameter strand.
2. For 0·128", 0·200" and 0·276" diameter wire.
3. For both wire and strand.

The jack works in conjunction with an instant-release grip, which permits rapid change-over on multiple work, and caters for long extensions (a second or third bite can be commenced within seconds of reaching maximum jack travel).

The set comprises self-retracting 6" or 12" extension jack, hand pump, pressure gauge, 6' length flexible hose with quick couplings, instant-release grip and jaws. These are all contained in a stout wooden box from which the pump may be operated without removal.

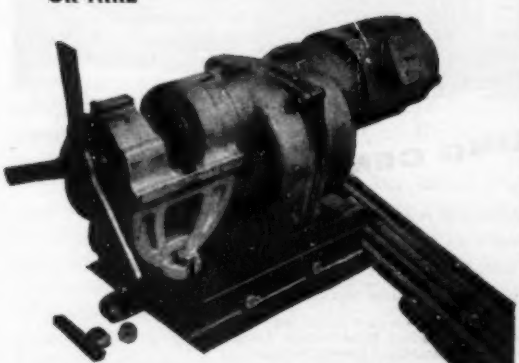
U.K. Ex-Works prices as follows:

	6" extension	12" extension
Complete Set for Strand Stressing	£95	£115
Complete Set for Wire Stressing	£87	£107
Complete Set for Both Wire and Strand	£114	£134
Conversion Set Strand to Wire	£19	
Conversion Set Wire to Strand	£27	

Send for our new booklet covering both pre- and post-tensioning.

STRESSING DEVELOPMENTS LTD.

95 KINGSTON HILL, KINGSTON-UPON-THAMES ***TELEPHONE:*** **KINGSTON 2372**



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CONCRETE AND CONSTRUCTIONAL ENGINEERING

INCLUDING PRESTRESSED CONCRETE

Volume LV, No. 1. LONDON, JANUARY, 1960.

EDITORIAL NOTES

Towards the Robot.

AT Queen's University, Belfast, recently, the Duke of Edinburgh again pleaded for the better education of technologists. In the course of his remarks he said:

"The conception of the university as simply a means to enhance the commercial prosperity of the State reduces higher education to the level of cattle breeding. So many square feet per student, so many years to mature, a course of technical instruction, graded by examination, packed in a diploma and delivered to industry free of charge, and another statistic joins the commercial battle. That way of thinking must be resisted at all costs. Unless higher education combines vocational training with the proper development of mind and character the universities are doing only half their job. The two great dangers of the commercial view of universities are that it encourages an unacceptably high degree of specialisation and that it produces men who are productive and useful only at the beginning of their careers instead of throughout the forty years of their expected working life. Science and technology will always be the controlling force in the nation's prosperity, but only so long as the manager-technologist has the benefit of a broad-minded traditionally liberal university education centred on his technical specialisation."

This is not the first time that His Royal Highness has commented on the shortcomings of our teaching system. In 1953, speaking at Edinburgh University, he dwelt on the "narrowness of mind which too many of the products of good schools and universities seemed to show", and emphasised the value to scientists and technologists of the more liberal education that imparts a broader outlook on specialised subjects. The Duke thus adds the weight of his authority to the views of many lesser men who have spoken and written in the same strain on a problem that is vital to the well-being of a nation that must increasingly depend upon its capacity for invention, which in turn depends only on powers of reasoning, to maintain its standard of living. There can, however, be no improvement without a higher standard of education of the students accepted by the universities and technical institutes. The educationists seem to be obsessed with the idea that the imparting to students of the rudiments of science and technology is more important than general education. This is largely the fault of those universities that have lowered the standard of education required of students

who sit for their entrance examinations, and refuse to educate the uneducated people whom they have admitted to their technological courses. London University has gone so far as to omit English as a subject in its examination for degrees in technical subjects, and we have the ridiculous—and alarming—state of affairs that a man who cannot express himself intelligibly in his own language can receive a degree from an institution that used to be known as a seat of learning. " Writing maketh an exact man " is as true now as when Bacon wrote it; lack of the ability to write clearly results in the loose thinking that is so prevalent amongst technologists—and their teachers—to-day. In most Continental countries the schools devote much more time to general education and less to science and technology, with the result that the students are better fitted to take advantage of the technical training offered by the universities and technical high schools.

It is possible that the changes in ideas of the purpose of education and the tremendous increase in the numbers undergoing technical training will prevent any improvement, and that uneducated technologists will in future be more common than they are to-day. Uneducated men now being awarded technical degrees (and even doctorates of philosophy) will be the lecturers and professors of tomorrow. Few of these men will be able to teach more than their own specialised subject, and their uneducated students will in turn be awarded degrees and become professors and lecturers; indeed this is already happening. In the schools the liberal arts are being taught less and less because the teachers know little or nothing of these subjects; recently the National Association of Schoolmasters stated that 29,000 of its members are " untrained ", which can mean only that they are unfitted to teach. As time goes on it seems inevitable that the standard of education will become progressively worse.

It does not seem likely that much will be done to implement the plea of the Minister of Education for liberal education in technical colleges. Among the members of the Association of Teachers in Technical Institutions strong opposition to this suggestion has been expressed in illogical statements such as: To assert that students engaged in vocational studies should in addition be compelled to attend non-vocational classes is essentially illiberal, and The chances of passing examinations may be materially reduced by presumptuous and misguided insistence on liberal studies. What could be more misguided than to suggest that education should not be compulsory in the case of a technologist? There is little doubt that such opposition is due to the fact that many of the teachers are as ignorant as their pupils of anything beyond the narrow rut of technicalities. And so we have a system in which instructors in schools and technical institutions cannot teach their pupils how to express themselves in their own language, and the universities accept such young people for technical courses and give them no further education. The result can only be future generations of uneducated technologists except for the few who have the will to educate themselves. It is also desirable that our leaders should set a better example of clear expression. It may be thought to be unfair that technologists should be blamed for using a language that is unintelligible to others when a Junior Minister can assure the House of Commons that he is " fully conscious " when he means that he knows, and a peer can inform the House of Lords that there is " unemployment in the steel industry due to shortfall of offtake ".

Design of a Helical Stair Built in 1908.

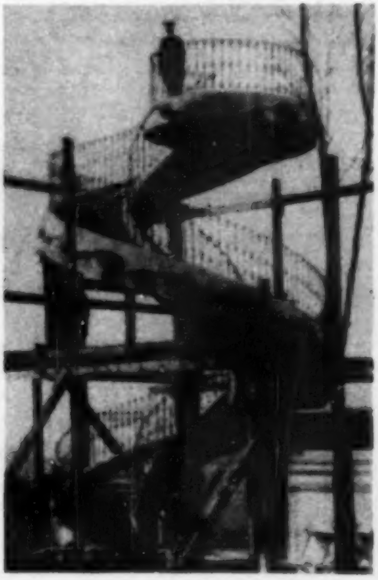
THE helical stair shown in the illustrations was built for use as an observation platform at the Franco-British Exhibition held at the White City, London, in 1908. The stair was designed by Messrs. L. G. Mouchel & Partners, by whose courtesy we are able to reproduce on the following pages the calculations and some of the drawings. It is seen that the calculations for determining the stresses and for estimating the quantities of concrete and steel are a remarkable contrast to the amount of arithmetic often used to-day to achieve the same result.

Reference is made in the calculations to the English formula for the relation between the load, the stresses, and the dimensions of a helical spring made of material of rectangular cross section. This formula can be written

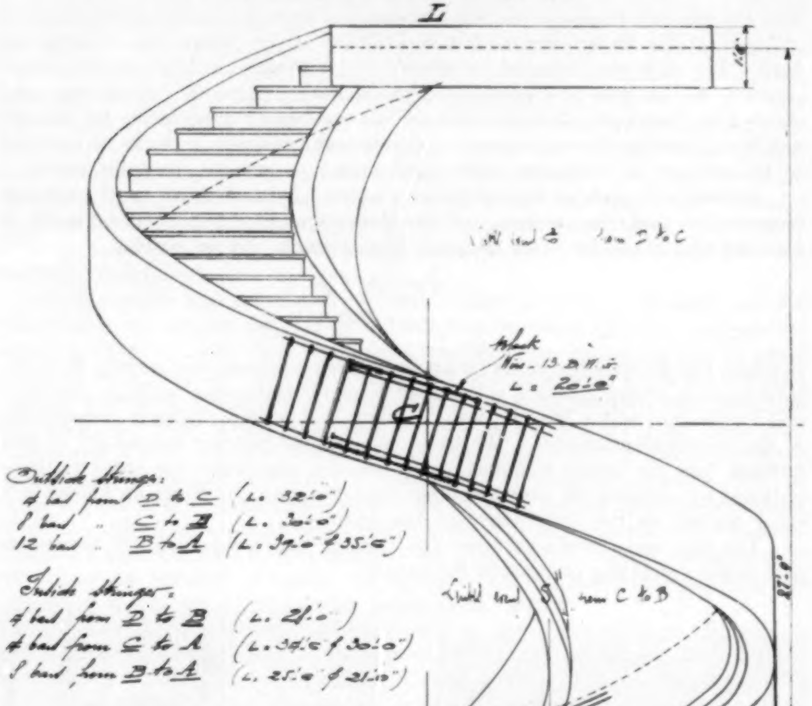
$$f = \frac{WR\sqrt{b^2 + d^2}}{0{\cdot}514 b^2 d^2}$$

in which f is the shearing stress (in kilogrammes per square centimetre), W is the total load (in kilogrammes), R is the mean diameter of the stair (11 ft. = 335 cm.), and b (162 cm.) and d (71 cm.) are the width and overall thickness respectively of the rectangular member. We have been unable to trace the source of this formula, but the results obtained by its use are practically the same as those obtained by applying the corresponding expression given by Professor Unwin and other authors in the early years of this century.

The stair was demolished when the London County Council built houses on part of the exhibition grounds in the 1930's.

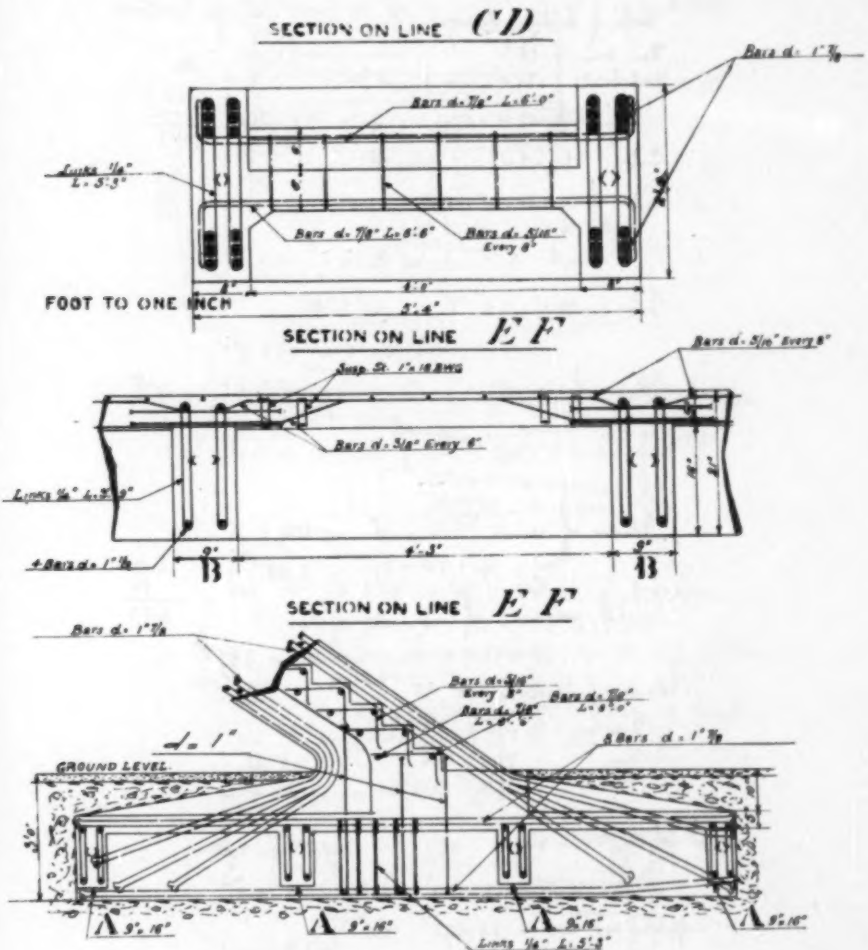


DETAIL FROM WORKING DRAWING.
(See also cross sections on facing page.)



Helical Stair Built in 1908. (*See page* 3.)

DETAIL FROM WORKING DRAWING.
(See facing page for positions of cross sections.)



Helical Stair Built in 1908. *(See page 3.)*

QUANTITIES AND LOADS.

2211

17 January 07

Spiral Staircase

Cube of outside stringer

Mean Diam = 15.8″

Total height = 27.0″ (from ground level).

Length $\sqrt{2.1445.66 + \overline{27}^2}$ + $\frac{3}{2}$ = 79′0″ (about)

432
2420
324
2744

Cube = 79 × 2′4″ × 8″ = 123 ft

145
310
119

Cube of inside stringer

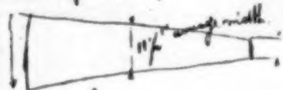
D. = 6′4″. Height 27′0″

Length √ × $\frac{3}{2}$ = say $\frac{3}{2}$ × 26.8 = 40.0″

Cube = 40′0″ × 2′4″ × 8″ = 62.5 ft

123 + 62.5 = 186 c ft.

Volume of one step 11½″ × 4′0″ × 3″ = 0.96 ft



Volume of concrete underneath one step

say 11½″ × 4.0″ × 6½″ = 2.08 ft

2.08
.96
3.04

Total volume of concrete of stair

123 + 62.5 + 53 × (0.96 + 2.08) = 347 ft

161.5

Volume of platform at top.

Area say 7′ × 12′6″ = 87.5

Floor 87.5 × 4″ = 29 ft

Beam 27′6″ × 8″ × 1′2″ = 21 ft

50 ft

Total volume 347 + 50 = 397 say 400 ft

Dead weight 400 × 70″ = 28 000″

Superload say 200″ on 53 steps 10600″
1 cwt per sq ft of platform = 87.5 × 51 = 4400 } 15.000

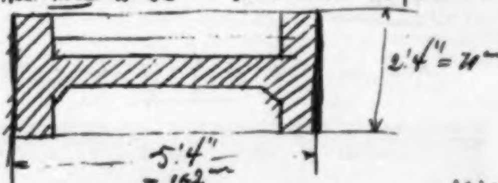
Load at Ground level = 43.000 lbs.

Parapet not included

Helical Stair Built in 1908. *(See page 3.)*

DESIGN.

Taking english formula for spiral spring made of rectangular wire and assuming the section of the "wire" to be 5'4" x 2'4" as per sketch



$$W = \frac{.514 f \times \frac{162^2 \times 71}{\sqrt{162^2 + 71^2}}}{335} =$$

26200
5040
31240
1.77

$$W = \frac{.514 f}{335} \times \frac{26200 \times 5040}{177}$$

$$f = \frac{47000 \times 335 \times 177}{0.514 \times 26200 \times 5040} = \frac{19.5}{.6.27} = \cancel{31.2}\ 40.6.$$

Total load on section $\cancel{31.2}$ 40.6 × 162 × 71 = ~~...~~ 47.000 kgs.

$$A = \frac{47000}{12} = \cancel{40000}\ 40000 \text{ cm}^2$$

6 ch 1"7/8

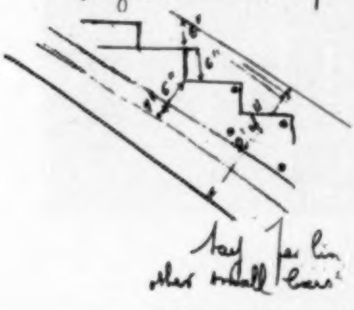
Say ~~8 to 1 5/8"~~ at top & bottom of each beam

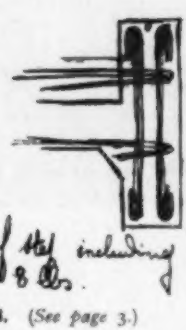
4 × 10717 = 43000 cm

4 of these bars are taken to the top of the spiral

Average weight per foot including links of say 5/16" or 1/4" every 6" or 8" 106 lbs per beam

2 Horizontal bars in steps 7/8"





Say per lin ft of step including other small bars 8 lbs.

Helical Stair Built in 1908. (*See page* 3.)

FOUNDATION.

Slab

Load on slab at footing say 43000 k

$\frac{33 \times 20}{256}$ Area of slab 16'0" × 16'0" = 256 □' 24 □m

Average load per □m on ground say 0.2

$\frac{43000}{240}$ = 1790 Kgs

To be on the safe side let us take 2000

Flooring 5" 3.75 ft × 6 m.

Beams Or each 2000 k × 1.60 = 3200 per l. m

9"×16" M: $\frac{3200 \times 3.35^2}{2}$ = 17000 mk

1 ft × 10"

Longitudinal beams under stringers.

18"×24" 3 ft × 40 kgs

Quantities 256' = 28 □ yds

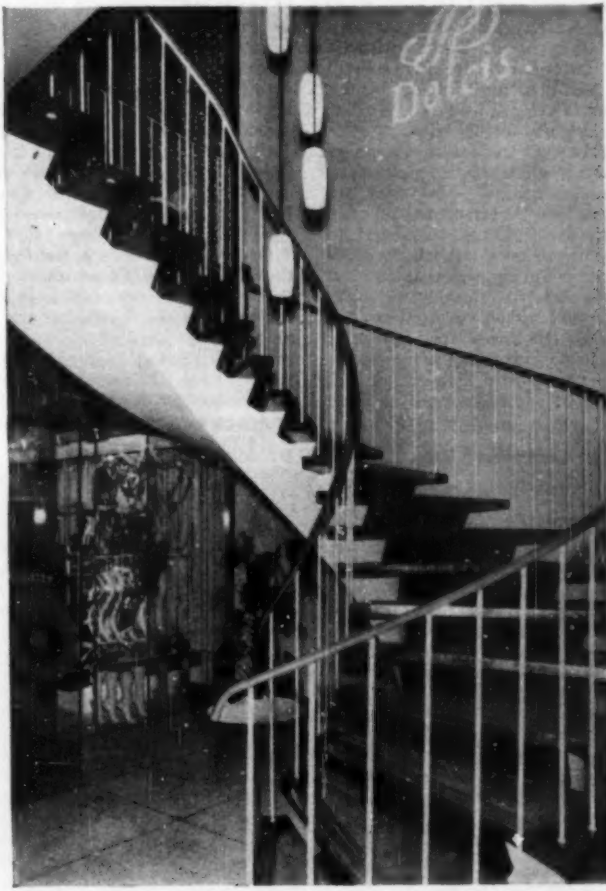
	C	S
Flooring	105	168
Beams 5 × 16 × 1 × 10	80	400
2 × 16 × 3	144	1920
	329	2888 kgs

12 ft a 10s per yd = 225 lb.

Helical Stair Built in 1908. (*See page* 3.)

A Helical Stair.

THE stair illustrated below is at a shop in Norwich. The flight is 11 ft. 9 in. high and 9 ft. 3 in. in diameter. The helical slab is 9½ in. thick, and is reinforced with fourteen mild-steel bars of ⅞-in. diameter and ½-in. links at 6-in. centres on the centre-line of the stair. In order to complete the work quickly high-alumina cement was used; seven days after the concrete was cast the shuttering was removed and the stair was used for light traffic, and full use of the stair was permitted seven days later. The architect of Dolcis, Ltd., Mr. Ellis E. Somake, F.R.I.B.A., was responsible for the work. The stairway was designed by Messrs. Malcolm Glover & Partners and built by Messrs. R. G. Carter, Ltd.



Competition for the Design of an Elevated Motorway.

In the competition arranged by the Prestressed Concrete Development Group for designs for an elevated motorway over an existing road the first prize of £500 has been awarded for the design submitted jointly by Mr. W. F. G. Crozier, Mr. J. D. Bennett, and Mr. C. G. Kerry. The second prize of £200 was awarded for the design submitted jointly by Mr. E. O. Measor and Mr. A. Bailey, O.B.E. Prizes of £100 each were awarded for the designs submitted by Mr. F. Graham, Mr. R. W. Hobbs, and jointly by Dr. Ulrich Finsterwalder and E. Paulus. There were twenty-five entries.

The requirements were that prestressed concrete, either alone or in conjunction with reinforced concrete, be used and that the structure be supported on piers over a strip dividing the two carriageways of the lower road on which traffic would continue during construction and after completion. A design was also required for part of the elevated road to be away from the lower road so as to permit a more economical arrangement for the supports; the area under this part of the road would be used as a car park.

The Assessors' Report

The designs placed first and second are illustrated on the facing page.

The assessors, who acted in a personal capacity, were Mr. H. C. Adams (Ministry of Transport), Mr. C. P. Andren (University College, London), Mr. A. G. Jury (City Architect, Glasgow), Lt.-Col. G. W. Kirkland (Past President, Prestressed Concrete Development Group) and Sir Herbert Manzoni (City Engineer, Birmingham).

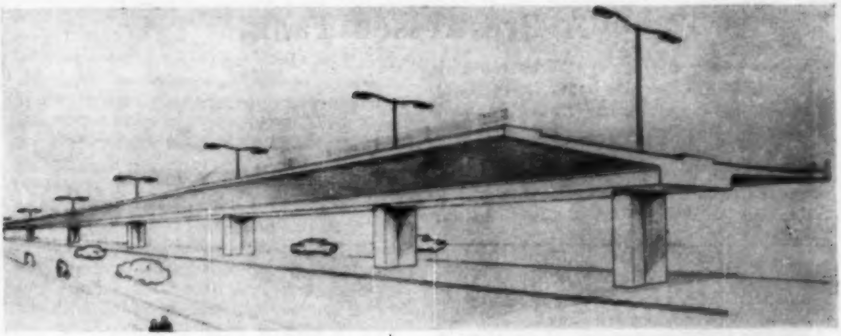
The assessors report that there was no outstanding design "which, while being æsthetically good, was also adequate in structural design, in arrangements for traffic flow and in ease of construction. The safety of pedestrians using the lower road has generally been ignored, and little thought has been given to the problem of expansion joints in the wearing surface. All the designs contain defects in one or more of these aspects. In some cases, however, the defects could be remedied or reduced without fundamental alteration to the main design."

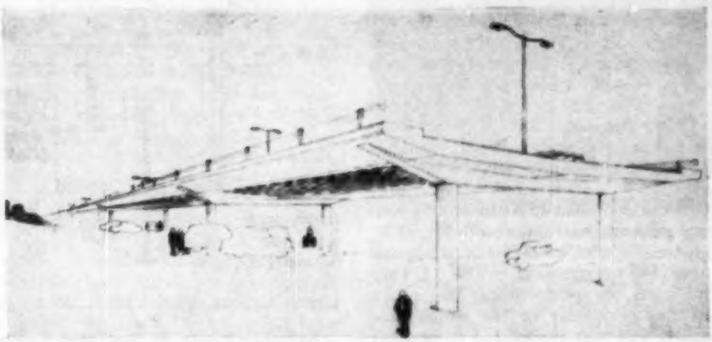
Design Placed First.—

In commenting on the winning design (*Figs.* 1 and 1a) the assessors state: This is æsthetically acceptable, structurally good, and provides a reasonable traffic solution for a speed of 50 miles an hour. The transverse members are designed for all possible conditions of loading. The stresses are satisfactory for all the cases considered, but there may be a critical plane about 7 ft. from the end of the cantilever. The central longitudinal beam would be prestressed after the concrete deck had been cast in place. The combined effect is not considered in the calculations, in which it is assumed that the entire prestress is in the central beam. The calculations are thorough and give a satisfactory analysis of the problems related to torsion and concentrated loads. The car-park section of the motorway is in reinforced concrete designed by the load-factor method.

Design Placed Second.—

In their comments on the design awarded the second prize (*Fig.* 2) the assessors state: The design is structurally sound and provides the best solution of the traffic problem. Its appearance is considered to be poor in that too much consideration appears to have been given to functionalism, the elements are over-elaborated, and the parapets appear to be unpleasantly heavy. The design is structurally simple, comprising prestressed I-beams longitudinally and a topping cast in place. The calculations generally are thorough, but high stresses would occur in some places and the losses of prestressing force allowed for in some of the members with pre-tensioned steel appear to be low. The construction method appears to be simple and satisfactory.





Figs. 1 and 1a.—Design Placed First. By Mr. W. F. G. Crozier, Mr. J. D. Bennett, and Mr. C. G. Kerry.

Fig. 2.—Design Placed Second. By Mr. E. O. Measor and Mr. A. Bailey, O.B.E.

A Prestressed Tank.

A NEW reservoir (*Fig.* 1) at Whatborough, Leicestershire, for the River Dove Water Board is a cylindrical tank having a capacity of 2,000,000 gallons, and comprises a reinforced concrete base slab 8 in. thick, a prestressed wall of 143 ft. internal diameter, 19 ft. 9 in. high, and 8 in. thick, and a prestressed domical roof 3 in. thick.

The joints at the top and bottom of the wall are shown in *Fig.* 2. It was calculated that a radial contraction of ¼ in. would occur when the wall was prestressed. To prevent the wall bending vertically, the joint at the base of the wall is flexible, and comprises two circumferential strips of rubber, each 1½ in. wide and ⅝ in. thick, separated by a PVC water-bar. The resistance of the rubber strips to horizontal shearing forces is known accurately and is small, so that lateral movement of the base of the wall is practically unrestricted. The joint between the roof and the top of the wall likewise permits movement of the wall.

High-tensile wire of 0·176-in. diameter was used for prestressing. The 2½ tons of wire used for prestressing the beam around the perimeter of the dome was galvanised and was embedded in cement mortar as it was being wound in a recess which was later filled with gunite. About 6½ tons of wire was wound around the wall of the tank which was then filled with water so that the gunite, which was applied in a layer 1½ in. thick, would not be subjected to tensile stresses.

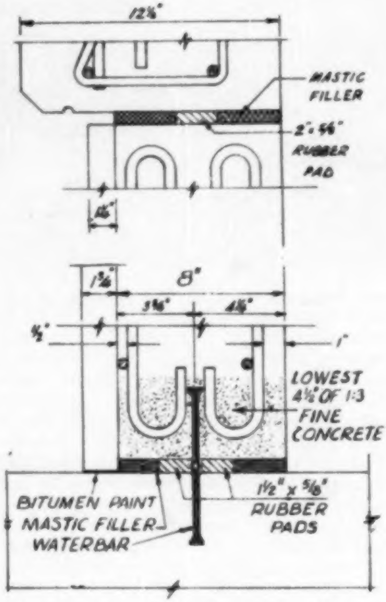


Fig. 2.—Joints at Top and Bottom of Wall.

The engineer of the River Dove Water Board is Mr. H. Wallhouse, and the tank was designed and constructed by Preload, Ltd.

Fig. 1.—The Tank Before Guniting.

The U.S.A. Embassy in London.

THE new building (*Fig.* 1) for the U.S.A. Embassy in Grosvenor Square, London, is a reinforced concrete structure containing some precast and cast-in-place construction. The main building is rectangular on plan and is 275 ft. by 143 ft., with entrance stairs projecting on the south, east, and north sides. The basement, the lower ground floor, which is at about the level of the pavement, the ground floor, and the first floor cover the whole of this area, and a sub-basement covers part of the area. Above the first floor the building is U-shape in plan and is of four stories. It is separated from the rear rectangular part by a joint ½ in. wide which is provided in case of settlement and extends into the foundations.

The foundations are mainly strip footings of plain concrete extended 3 ft. into clay, with reinforced concrete distributing beams which are formed either separately or by thickening the floor of the sub-basement. All the columns were cast in place. The floors of the sub-basement and basement are mainly of reinforced concrete 9 in. thick. Around the periphery of the rectangle there is a 9-in. retaining wall coated on the outside with asphalt which is protected by 4½-in. brickwork. The wall extends to the lower ground floor, where it is thickened to form the base for cruciform columns which extend to the first floor. At the ground floor, the beams which carry the ground floor span between the columns.

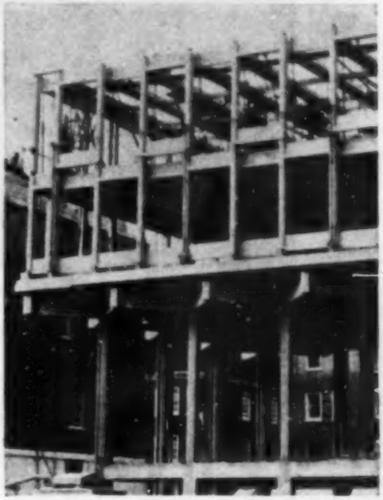


Fig. 2.—Construction of Walls above First Floor.

Above the first floor the construction consists basically of three reinforced concrete towers which extend from the foundations and are connected with one another by a cast-in-place beam-and-column "spine" construction which supports the upper floors. The central tower contains two lift shafts with 5-in. reinforced concrete walls cast in place. The

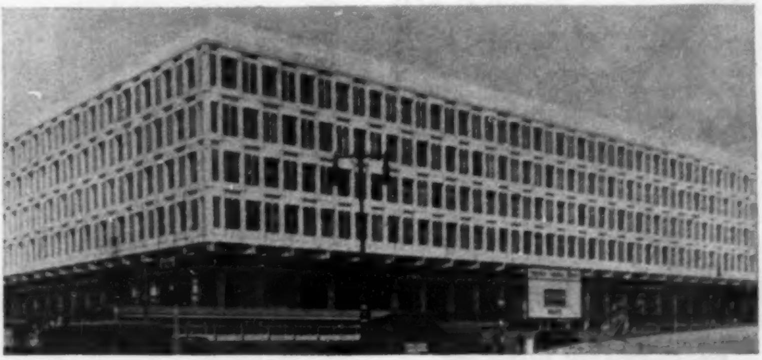


Fig. 1.

Fig. 3.—Cantilevered Diagonal Beams at First Floor Level.

other towers are constructed similarly but contain stairs, lavatories, and services.

The reinforced concrete columns in the lower part of the structure are mainly at 11 ft. centres and, with the reinforced concrete towers, support the diagonally-intersecting reinforced concrete beams of the first floor (*Fig.* 3). The beams, which are at 7 ft. 9½ in. centres in both directions, were cast in permanent shuttering formed of precast concrete (*Fig.* 4). The shutters are 2 ft. 2 in. deep and 11 in. wide; the sides taper from 1½ in. thick at the bottom to 1 in. thick at the top. There are castellations in the bottom of the shutter. The reinforced floor slab cast in place over the beams is generally 4 in. thick. For about half the area of this floor, a space of 1 ft. is provided between the top of the beams and the underside of the slab to accommodate services. For this area 2-in. precast slabs were used as permanent shuttering on which was cast a 2-in. topping. These shutter slabs were cast on the site and placed in position after the services had

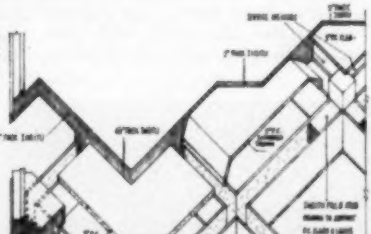


Fig. 4.—Diagonal Beams and First Floor Slab.

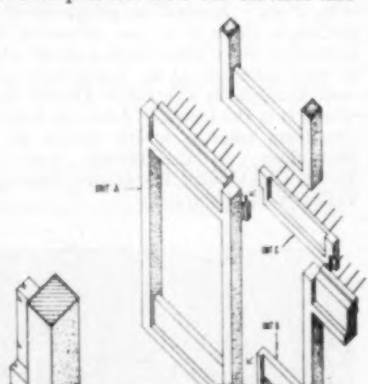


Fig. 5.—Details of Wall Frames.

been installed. Where ducts were not required a precast member having the shape of an inverted U was used instead of the flat slabs.

The diagonal beams cantilever about 5 ft. from the face of the columns (*Fig.* 2) and carry at their ends the precast concrete wall frames, which support the loads from the upper floors and are provided around the entire periphery of the building. The wall frames are rectangular and

Fig. 7.—Temporary Struts for Wall Frames.

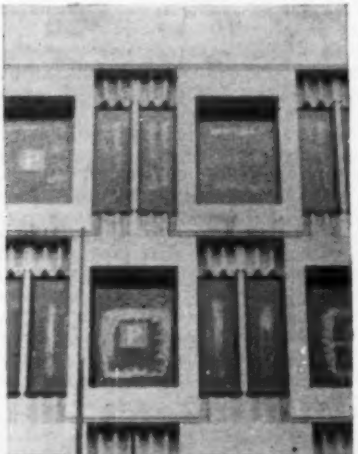


Fig. 6.—Part Elevation.

each comprises two 8½-in. by 7-in. columns of one-story height connected at the top and bottom by a lintel and a sill respectively (*Fig.* 5). The frames are at 11 ft. centres which results in the columns being at 5 ft. 6 in. centres. The frames in any story are staggered in respect to the frames in the stories immediately above and below (*Fig.* 6). Two threaded mild-steel bars project from the bottom of each column in such a position that they fit into holes in a plate on the frame below. Nuts and washers are fitted above and below the plate and, after adjustment, the connection is completed by filling the remaining space with concrete. At the top of each column, on the inner face, there is a nib which supports the floor beam and projects sideways to support the lintel. A member similar to the lintel forms the sill and is bolted to the back of the frame. The temporary strutting used for aligning the wall frames is shown in *Fig.* 7.

The upper floors comprise I-beams at 5 ft. 6 in. centres. Prestressed precast concrete planks 4 in. thick form the bottom flanges of the beams (*Figs.* 8 and 9). Castellations are formed on the top of the planks to form a key for the upper

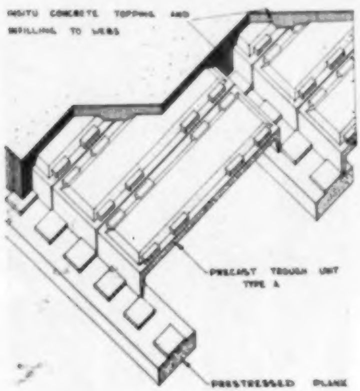


Fig. 8.—Details of Upper Floor up to 1 ft. 3 in. Thick.

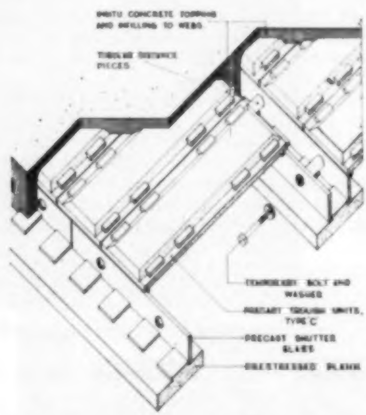


Fig. 9.—Details of Upper Floor of 1 ft. 7½ in. and Greater Thickness.

part of the beam, which is cast in place. The top flange of the beam is formed by the floor slab, which consists of a reinforced concrete topping generally 1½ in. thick laid on inverted precast concrete troughs serving as permanent shuttering. In the north-east and south-east corners, where there are junctions of beams at right-angles to each other, there are Y-beams each of which consists of a straight precast beam and a cranked beam forming the two arms. The straight part of the Y-shaped beams is prestressed. The tops of the beams are castellated to bond with a 4-in. concrete topping cast in place. The overall depth of beam and slab is 17½ in.

At the rear of the area between the two wings of the U-shape building there is a roof, at the level of the first floor, which is supported by prestressed concrete beams at 11 ft. centres spanning 55 ft. The wires in the beams were post-tensioned. The beams were cast monolithically with the roof slab and there are four large holes through each for services. The slab was cast on inverted precast troughs between the beams; the beams are so designed that a four-story extension can be erected upon them at a later date.

The architects are Messrs. Eero Saarinen & Associates in association with Messrs. Yorke, Rosenberg, and Mardall. The consulting engineers are Messrs. Felix J. Samuely & Partners. The general contractors are Messrs. Pauling & Co., Ltd., the sub-contractors for the structural concrete are Messrs. Wates, Ltd., the precast members were made by the Modular Concrete Co., Ltd., and the prestressed planks by Messrs. Cawood Wharton & Co., Ltd.

Impurities in Sand.

The result of an investigation, made at the Finnish State Institute of Technical Research by T. Karttunen and T. Sneck, on the effect of humus in sand on the strength of concrete is given in "Inverkan av Humushaltig Sand på Betongens Hållfasthet". It is concluded that the usual colorimetric test for the cleanliness of sand is unsatisfactory because it does not indicate the chemical properties of impurities. Some sands that produced a dark colour in sodium hydroxide were found to be harmless whereas some sands that did not change the colour of sodium hydroxide would be harmful if used in concrete. It is suggested that inorganic salts in humus may act as accelerators in the presence of alkali.

A College at Manchester.

The construction of the new Domestic and Trades College at Fallowfield, Manchester, is almost complete (*Fig.* 1). A photograph of a model of the buildings was given in this journal for January 1959.

The college comprises five buildings. The two-story catering and administration building is the shape of a U on plan. The areas within the sides of this building are for use as car parks, above part of which is an assembly hall with a domed roof and supported at the centre by a column with a flared head. Across the open end of the U is the six-story teaching building, beyond which is a gymnasium and a tailoring building.

The ground consists of 1 ft. of made-up ground and top soil below which is a stratum 20 ft. thick of stiff boulder clay overlying bunter sandstone. Tests of samples obtained from boreholes indicated that the foundations of the buildings should be on the boulder clay and that the net pressure applied should not exceed 2·4 tons per square foot. Generally, the underside of the foundations are at a depth of 4 ft., but in the case of the teaching building the depth is 5 ft. Any excavation to remove weak ground below these depths was filled with plain concrete. Each building is constructed as a separate structure, and at points of interconnection provision is made for possible differential settlement, which is calculated to be about 1 in. between the teaching building and the catering building, and 1½ in. between the Assembly hall and the catering building.

The buildings are of reinforced concrete cast in place; the crushing strengths of the concrete at 28 days is not less than 3300 lb. per square inch for the foundations and 4500 lb. per square inch for the superstructures. The maximum and minimum limits of the cement content were also specified. Owing to the restricted site, ready-mixed concrete was used throughout. The arrangements for the distribution of concrete to the various parts of the works were such that it could be placed in the shuttering within about 45 minutes after mixing. The decision to use high-grade concrete was governed by the desire to reduce the sizes of the members as much as possible and, since much of the concrete is exposed, there was the added advantage of having concrete of good

Fig. 1.—View during Construction.

quality for appearance and resistance to the weather.

Multiple-story Building.

The overall height of the six-story building, on the roof of which there is a garden, is 120 ft. and the length is 221 ft. The building is shown in course of construction in *Fig.* 2, and a cross section is given in *Fig.* 3. The width between the inner faces of the inclined columns of the arched frames, which are exposed as a main architectural feature of the building, tapers from 69 ft. at the ground floor to 46 ft. 8 in. at the third floor and 28 ft. at the sixth floor. There are two lifts to the fifth floor and two stairs at each end of the roof garden. The inclined columns are 3 ft. by 1 ft. 3 in. in cross section and are at 10 ft. centres. An intermediate row of columns is provided up to the third floor, above which all the stories are free of internal structural columns. The foundations of the external and internal columns are continuous strip footings bearing on the boulder clay. The outward thrust of the inclined columns is resisted by transverse reinforced concrete ties which are provided at 20 ft. and 30 ft. centres, the strip footings acting as horizontal beams spanning between the ties. The walls between the inclined columns are faced with red clay tiles.

The floors are of beam-and-slab construction. The first floor, on which are the bakeries, has a 6-in. reinforced concrete slab designed to carry a general imposed load of 168 lb. per square foot, but in the bakeries the design load is 336 lb. per square foot to allow for heavy equipment being installed in any position. The second to fifth floors have 4-in. slabs, and are designed for an imposed load of 60 lb. per square foot. The sixth and seventh floors have 6-in. slabs and are designed for an imposed load of 100 lb.

Fig. 2.—Tall Building during Construction.

per square foot. Architectural requirements limit the overall thickness of the floors to 2 ft. The total depth of the beams and slab is 1 ft. 10 in. There is 1½-in. finish on the floors and a ceiling suspended below the beams. Pipes for services are contained in the cavity between the ceiling and the underside of the slab. The beams and slabs were cast in one operation.

Assembly Hall.

The assembly hall, a cross section of which is shown in *Fig.* 4, is circular in plan. The first floor has a diameter of 80 ft. and is supported around half of its circumference on projections on the columns of the catering and administrative building, on three separate columns under the other half, and on a circular central column. The diameter of the central column is 3 ft. 6 in. at ground level, and increases to 5 ft. 2 in. at a height of 9 ft. 6 in., above which it flares out and merges into the slab of the first floor. The slab is designed as a circular plate. Depending on the relative settlement of the central column, which is carried on a separate foundation, the top of the slab may be in either tension or compression.

The roof over the hall is a stressed-skin aluminium dome of 72 ft. 6 in. diameter, supported on a reinforced concrete ring-beam with which a drain channel is combined. The ring-beam is supported on fourteen reinforced concrete columns which are 9 in. square and are supported on the floor slab.

Two tower cranes were provided by means of which all parts of the site could be reached. One crane was erected within the U of the catering and administrative building, and the other at the side of the teaching building near the tailoring building. The cranes travelled on rail-tracks, so that skips of concrete, shuttering, reinforcement, and other materials could be placed where required.

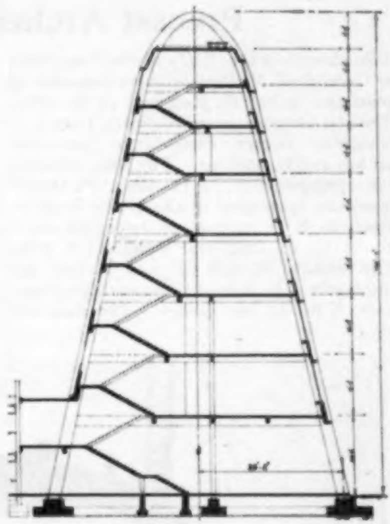


Fig. 3.—Cross Section of Tall Building.

The architect is Mr. C. Howitt, City Architect of Manchester. The consulting civil engineers are Messrs. L. G. Mouchel & Partners. The main contractors are Messrs. J. Gerrard & Sons, Ltd.

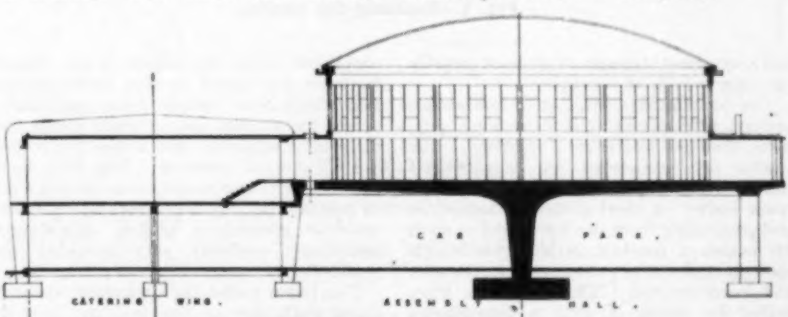


Fig. 4.—Cross Section through Assembly Hall.

Precast Arches for a Church.

THE church of Our Lady of the Visitation, at Greenford, Middlesex, is rectangular on plan and is 117 ft. long and 70 ft. wide. The transverse frames (*Figs.* 1 and 2) comprise twelve three-hinge parabolic arches at 8 ft. centres. The span between the springings is 74 ft. and the height from the springing to the crown is 48 ft. Each arch is precast as two ribs, each about 60 ft. long and weighing 6 tons. The breadth of each rib is 9 in. and the thickness 2 ft. 6 in. at the springing and 1 ft. 6 in. at the crown. The concrete had a crushing strength of at least 6000 lb. per square inch at six days.

The foundations (*Fig.* 3) of the arches are concrete blocks 11 ft. 6 in. long, 4 ft. wide, and not less than 4 ft. deep. The blocks are connected by longitudinal beams which support the exterior 18-in. brick walls. A steel dowel embedded in and projecting from the lower end of each rib enters a conduit in a recess which was grouted under pressure after the ribs had been erected. The joint was then sealed by filling it with a bituminous compound.

At the crown (*Fig.* 2) of each arch a dovetail slot is formed in the curved ends of the ribs. A $1\frac{1}{4}$-in. mild-steel dowel 1 ft. long was placed in the slot after a pair of ribs was erected. In one rib $1 : 1\frac{1}{2} : 3$ concrete was packed around the dowel and consolidated by ramming. The other end of the dowel enters a conduit embedded in the slot in the end of the adjacent rib, and therefore the joint is free to act as a hinge.

The ribs are shown in course of erection in *Fig.* 1. The arches are stabilised permanently by means of precast trough-shape members (*Fig.* 2), which were bolted between the arches as they were erected and which later formed permanent shuttering for a beam cast in place under the clerestory windows, the frames of which are of precast concrete. The first arch to be erected was maintained temporarily in position by guy-ropes and was used to stabilise succeeding arches. Additional temporary stability was provided by scaffolding.

The beam under the clerestory window along each side of the building is continuous and was cast around reinforcement projecting from the trough members.

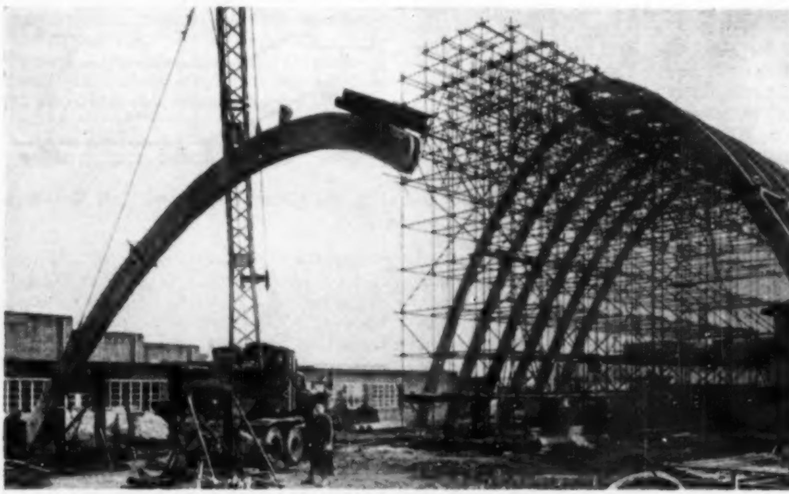


Fig. 1.—Erecting the Arches.

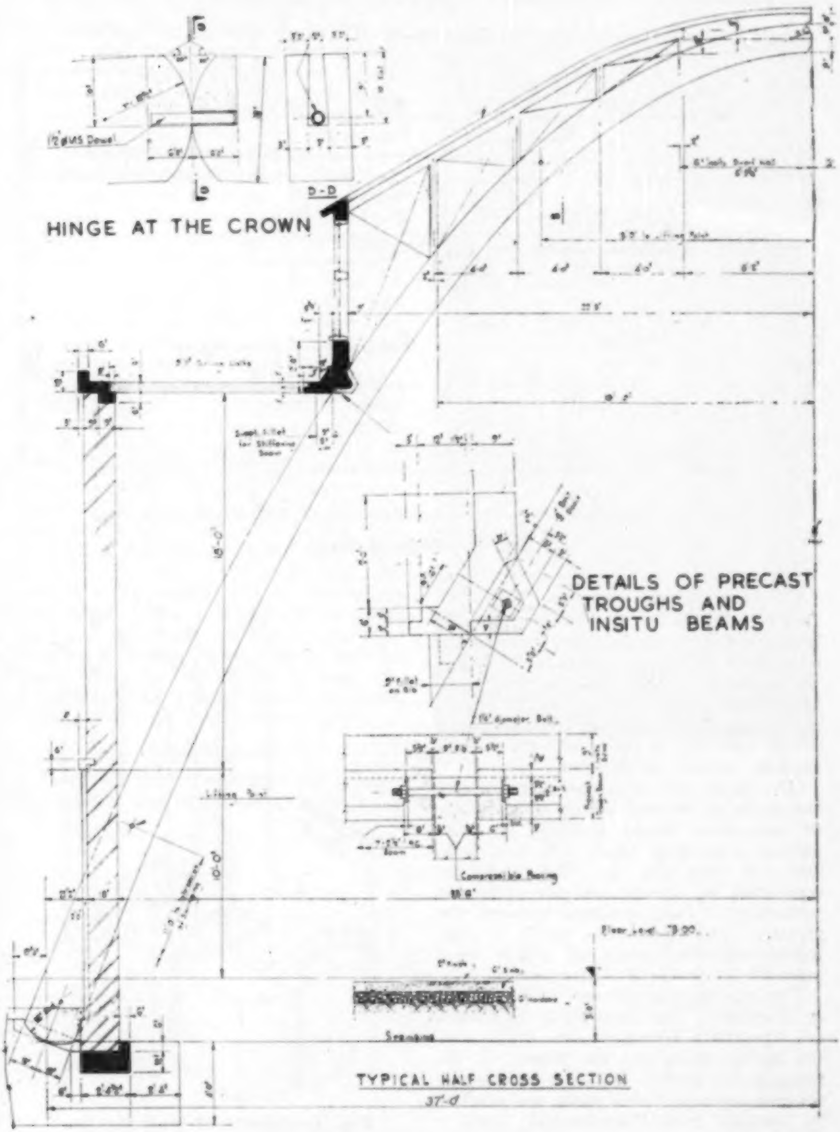


Fig. 2.—Part Cross Section.

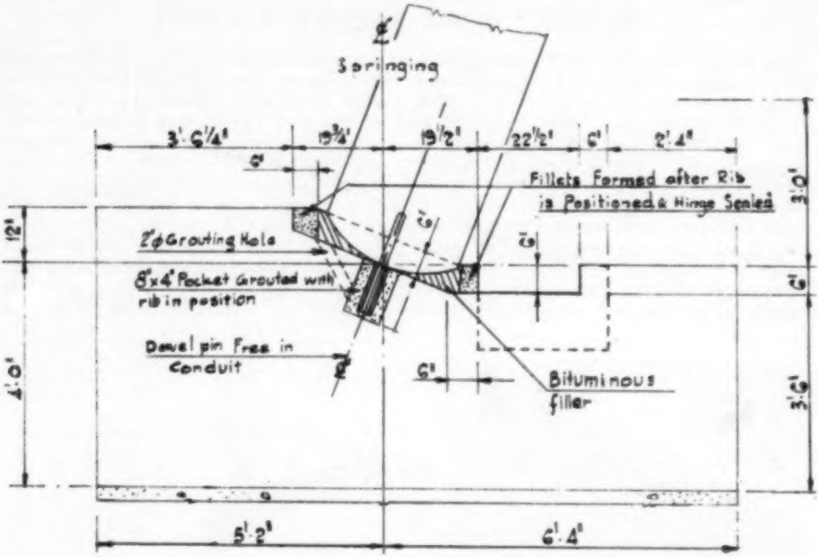


Fig. 3.—Hinge at Base.

Precast planks 6 in. thick spanning between this beam and the brick walls form the roofs at the sides of the building. The covering of the main roof (*Fig.* 2) is of precast planks carried on the 5-in. beams supported on 6-in. by 4-in. posts, which were cast in place with their lower ends in pockets in the ribs.

The north wall over the entrance and the walls at the end of the transept are of cast-stone blocks U-shape in cross section, 1 ft. 1½ in. thick, 1 ft. 6 in. wide and 3 ft. long (*Fig.* 4). The blocks are connected by dowels and are staggered vertically to form mullions between the glazing. The blocks are set in a reinforced concrete frame to which wind pressure on the wall is transmitted.

The architects are Messrs. David Stokes & Partners. The consulting engineers are Considère Constructions, Ltd., and the main contractors are Messrs. E. H. Burgess (London), Ltd. The precast concrete members were made and erected by Girlings' Ferro-Concrete Co., Ltd.

Fig. 4.—Details of Wall Blocks.

Water Tower of 150,000 Gallons Capacity.

THE construction of a water tower (*Fig.* 1) of 150,000 gallons capacity was completed in 1958 at a fertiliser factory for Messrs. Fisons, Limited, at Mucking, near Stanford-le-Hope, Essex. A requirement was that the bottom of the tank should be 70 ft. above the ground. Four preliminary designs (*Fig.* 2) were considered from the points of view of æsthetics and cost. The relative estimated cost of structures of the four designs were (a) 1·00, (b) 1·06, (c) 1·21, and (d) 1·32. Although design (c) might be the most striking in appearance it was thought that the additional cost was not justified, and design (b) was accepted as being economical and æsthetically satisfactory.

The structure comprises a cylindrical covered tank of 46 ft. internal diameter and 19 ft. deep supported on six columns equally spaced around the periphery of the tank and a central hollow column of 8 ft. diameter. The radial beams under the floor were cast monolithically with the outer columns and extend from the outer columns to the central column, where they are 3 ft. 6 in. deep. The height from the ground to the underside of the floor of the tank is 69 ft., and the depth of water 16 ft. A reinforced concrete helical stair is provided within the hollow column by means of steps cantilevering from the wall; within the column are also the inlet, outlet, overflow, and rainwater pipes.

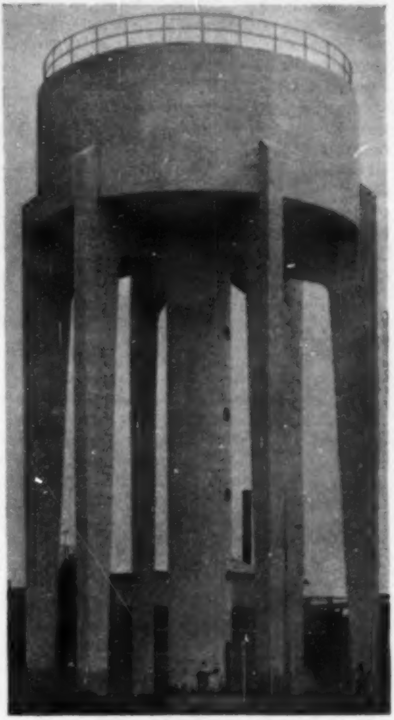
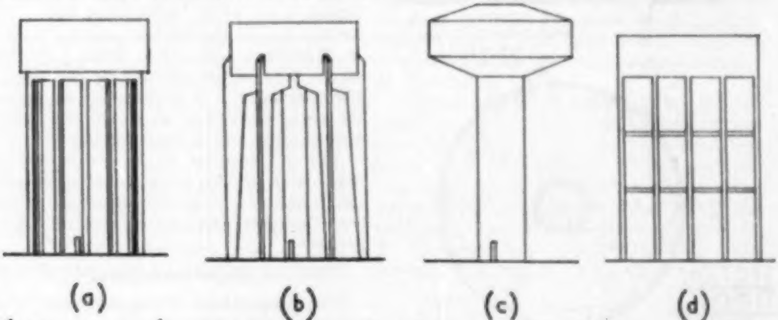


Fig. 1.

Foundations.

When the site was levelled the exposed ground was stiff brown clay to a depth of



(a) (Circular Tank) (b) (Circular Tank) (c) (Circular Tank) (d) (Square Tank)

Fig. 2.—Preliminary Designs.

8 ft. 9 in. overlying stiff blue clay. Triaxial compression tests on samples of the undrained brown clay indicated an average shearing strength of 2560 lb. per square foot, and a reasonable bearing pressure was considered to be 2¼ tons per square foot. With this pressure, consolidation tests indicated probable settlements as follows. Under the central column: initial settlement during construction, ½ in.; total settlement after forty years, 1¾ in. Under the outer columns: initially, ½ in.; after forty years, 1¼ in. In view of the magnitude of these

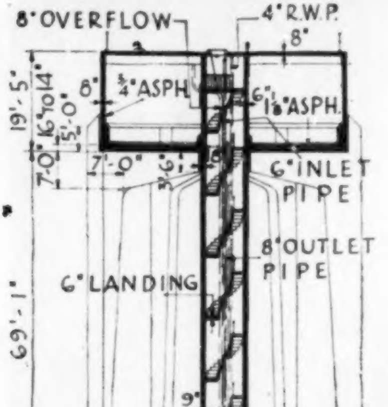


Fig. 3.

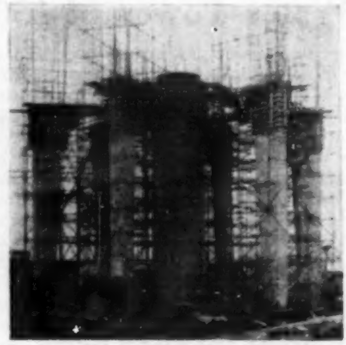


Fig. 4.

settlements the design pressure on the ground was reduced to be nominally 1½ tons per square foot.

Separate bases are provided under each outer column. Under static load the pressure on the ground does not exceed 1·5 tons per square foot under the outer column bases and 1·25 tons per square foot under the base of the central column. Due to the effects of wind the calculated pressures are 1·6 tons and 1·5 tons per square foot respectively.

The actual settlement on completion of the structure and with the tank full were as follows: ⅛ in. under the west and south-west outer columns, 1/16 in. under the north-west outer column, 3/16 in. under the north-east outer column, no settlement under east and south-east outer columns, and ¼ in. under the central column.

The base of each outer column is 14 ft. square and 3 ft. 6 in. deep and is reinforced with 1⅛-in. mild steel bars. The foundation of the central column is 27 ft. in diameter, and varies from 5 ft. deep at the base of the column to 2 ft. deep at the perimeter. It is designed as a circular plate subjected to a central concentrated load and a bending moment.

The SO_3-content of the soil is 0·02 per cent. and of the ground-water 39 parts per 100,000. Sulphate-resistant cement was therefore used for all concrete below ground.

Superstructure.

The proportions of the concrete in the columns and tank are 1 : 2 : 4 complying with B.S. Code No. 114. The reinforcement comprises mild steel bars, the

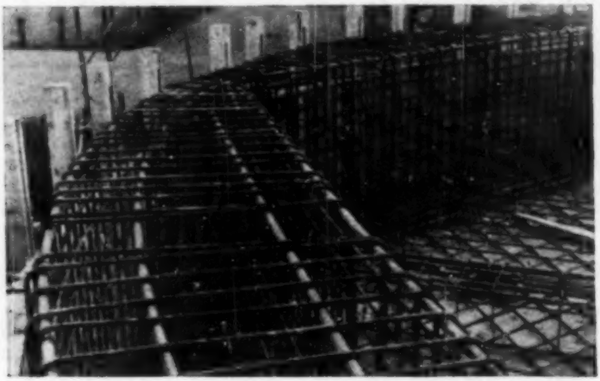


Fig. 5.—Reinforcement for Beam at Base of Tank Wall.

maximum tensile stress in which is 20,000 lb. per square inch. The tank is lined with asphalt $\frac{3}{4}$ in. thick on the wall and $1\frac{1}{4}$ in. thick on the bottom. The structure is designed to resist the wind pressures recommended in B.S. Code No. 4, Chapter V. The effects of wind are divided between the central column and any two outer columns in proportion to the stiffnesses of these members.

The six outer columns are 27 in. wide; in the other direction they taper from 7 ft. at the top to 3 ft. 6 in. at the base. The internal diameter of the central column is 8 ft. and the thickness of the wall is 9 in. For design purposes each outer column and beam are assumed to be pinned at the central column and fixed at the base. The main reinforcement in each of the outer columns, which are shown in course of construction in *Fig.* 4, comprises sixteen $1\frac{1}{4}$-in. bars with $\frac{3}{8}$-in. triple binders. The continuation of each column vertically outside the tank is an architectural feature having no structural purpose and is separated from the wall by $\frac{1}{2}$-in. fibre-board to allow for expansion.

The floor of the tank is designed as a series of triangular panels of 15-in. average thickness, supported by the outer columns and by a peripheral beam, at the base of the wall, which is also supported on the outer columns. The principal reinforcement in the slab comprises $\frac{5}{8}$-in. bars at 9 in. centres in each of three directions. The peripheral beam, the reinforcement in which is shown in *Fig.* 5, is designed to resist torsion in addition to ordinary design requirements, because it is segmental in plan and the centroid of the loading is not on the centre-line between the supports of the beam.

The wall of the tank is 8 in. thick and is reinforced to resist circumferential tension. Pipes passing through the wall of the tank are provided with puddle flanges embedded in a thickened part of the wall. A lead collar 6 in. greater in diameter than the pipe is sweated to the pipe and dressed against the first coat of asphalt on the inner face of the wall. The second coat covers the collar and is combined with a 2-in. fillet of asphalt and an asphalt sleeve 6 in. long surrounding the pipe. The flat roof is 8 in. thick and is designed to span radially from the central column to the wall.

The structure contains 774 cu. yd. of concrete and 65 tons of reinforcement, the ratio being 190 lb. of reinforcement per cubic yard of concrete. The structure was constructed in about seven months and cost about £25,000. The consulting engineers were Messrs. Brian Colquhoun & Partners, and the contractors were Messrs. Tileman & Co., Ltd.

A Hyperbolic-Paraboloidal Roof.

A GARAGE in Lincoln is 96 ft. by 106 ft. 9 in. on plan, and has a roof composed of two pairs of hyperbolic-paraboloidal slabs (*Fig.* 1). Each pair is separated by a strip 9 ft. 9 in. wide containing glazing and ventilators. The slabs are arranged so that their highest points are at the four corners of the building and on the centre-line parallel with the longer side.

Each hyperbolic paraboloid is 47 ft. 6 in. square on plan and is generally 2½ in. thick increasing to 9 in. at the edges (*Fig.* 2). The external edges are stiffened by upstanding beams 1 ft. wide and 1 ft. 6 in. deep. The internal edges are supported on three upstanding beams each 1 ft. wide and 2 ft. 6 in. deep, one on each side of the glazing and spanning 95 ft. and one intersecting them at right angles and spanning 105 ft. 9 in. The shorter beams are supported by V-shaped reinforced columns and the longer beams by rectangular columns 2 ft. by 1 ft. in cross section. Tubular steel columns 6½ in. in diameter are provided at the four corners of the building to prevent vertical movement.

The horizontal thrusts exerted by the slabs are each about 100 tons and act diagonally, and are resisted by four precast prestressed ties 10 in. wide and 1 ft. 6 in. deep; these are 15 ft. above ground-floor level and span 67 ft. The ties were precast in lengths of 60 ft. 6 in. and each contains sixty 0·276-in. wires and twelve deformed bars of high-tensile steel 1 in. in diameter. The wires and bars protruded 3 ft. at both ends to be embedded in the concrete of the roof.

Tubular scaffolding and timber shuttering were used for each pair of slabs; one slab was cast at a time and required two days for casting. On completion of the first pair of slabs the scaffolding and formwork were re-erected for the second pair. The two highest points are at the middle of the roof and these were supported on a steel tower until the structure was complete. The cost of the structure, excluding the foundation, was £7 10s. per square yard of area covered.

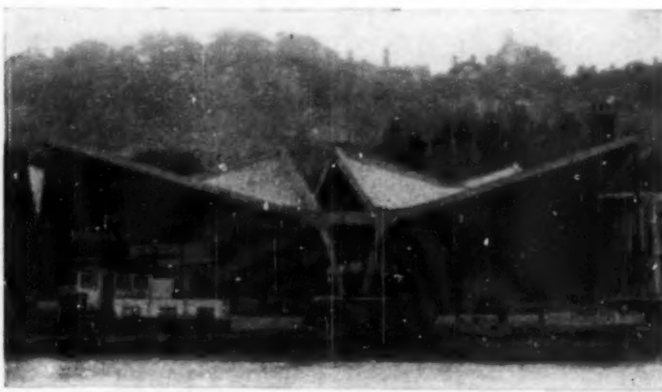
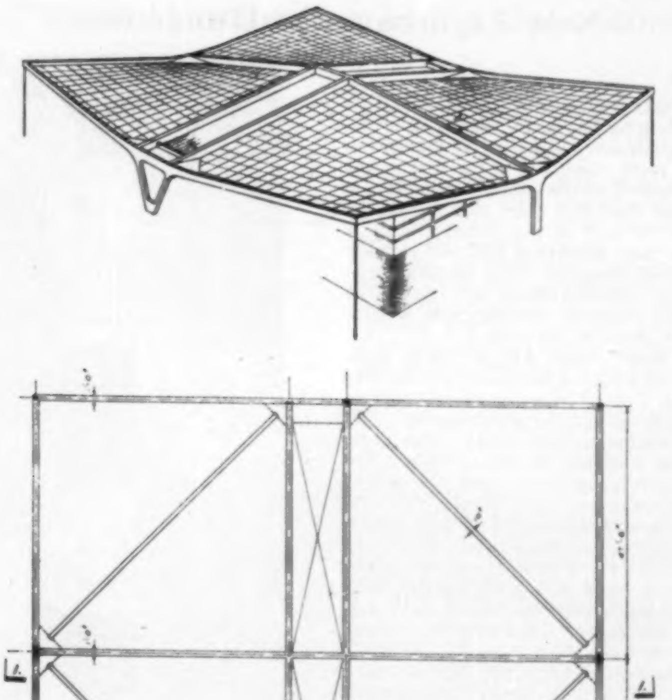


Fig. 1.

The architects are Messrs. Dennis Clarke Hall, Sam Scorer and Roy Bright, the consulting engineer is Dr. K. Hajnal-Kónyi, and the main contractors were Messrs. Gee, Walker & Slater, Ltd. The prestressed tie-beams were made by Messrs. Cawood Wharton Co., Ltd.



SECTION 1.1.

Fig. 2.—Details of Roof.

New Lighthouse at Dungeness.

THE illustration, which is reproduced from a painting by Mr. A. H. Walker, is of the new lighthouse being built at Dungeness on the Kent coast. It is the first major lighthouse to be erected by Trinity House for more than fifty years and will be the first of its kind in the country, since it will be fully automatic and will embody new ideas resulting from investigations made by Trinity House in co-operation with the architects and engineers, Messrs. Ronald Ward & Partners.

The tower, which will be 130 ft. high, will be on a base of white-cement concrete forming a spiral ramp which will enclose the machines and the control-room. The overall diameter of the tower will be 12 ft. and the wall will be 6 in. thick. The tower will comprise a number of superimposed concrete rings each 5 ft. high, and will be prestressed with high-tensile steel wires extending throughout its height. The bands of black and white will be in black and white cement with suitable aggregates, so that painting will not be necessary. In the topmost white band there are sixty holes for the loudspeakers which form the fog-signalling device, which will sound automatically when a fog warning is given by the fog-detector at the top of the ramp. The light for the main beam will be provided by a lamp which will be little larger than a domestic lighting bulb but will exceed 600,000 candle-power. The contractors are Taylor-Woodrow Construction Co., Ltd.

"Concrete and Constructional Engineering" Prize Design.

THE prize of £25 awarded each year by this journal for competition amongst the students attending the post-graduate course of concrete technology at the Department of Civil Engineering of the Imperial College of Science and Technology, University of London, has been awarded for the session 1959 to Mr. I. H. Reith. The competition was for designs for a port at Pimlico on the River Thames in accordance with conditions set by Professor A. L. L. Baker, the Professor of Concrete Technology.

The assessor was Mr. Leslie Turner, M.I.C.E., P.P.I.Struct.E., who in his report states : More than half of the forty entries were of such a high standard that the task of adjudication was difficult, and many of the drawings were exceptionally neat. Mr. Reith's design was for a jetty and warehouse, and was awarded the prize because it was consistently excellent in all respects.

Precast Construction in Multiple-Story Buildings.

The construction of the multiple-story buildings at the College of Technology at Derby (*Fig.* 1) and the Civic College at Ipswich (*Fig.* 2) is alike in that the external walls have precast frames, lattice girders are provided to support the first floor and the structure above, and the floors are mainly of precast members.

The building at Derby has a main part 115 ft. high, 123 ft. long, and 70 ft. wide, in which there are a ground floor, eight stories of classrooms, and a penthouse. The central part of this structure is 62 ft. by 17 ft. in plan and is of reinforced concrete beam-and-slab construction cast in place; it contains the stairs, which are precast, the lifts, and the chimney from the heating chamber. The reinforced concrete frame of this part resists the wind forces.

The two teaching buildings at Ipswich are each 117 ft. high and have floor areas of 63,000 sq. ft. and 27,000 sq. ft. respectively, and are connected by a structure containing the stairs and lifts. Wind is resisted by the floors acting as beams which transmit the forces transversely to concrete walls cast in place at the ends of the buildings, and longitudinally to columns cast in place in the middle of each building.

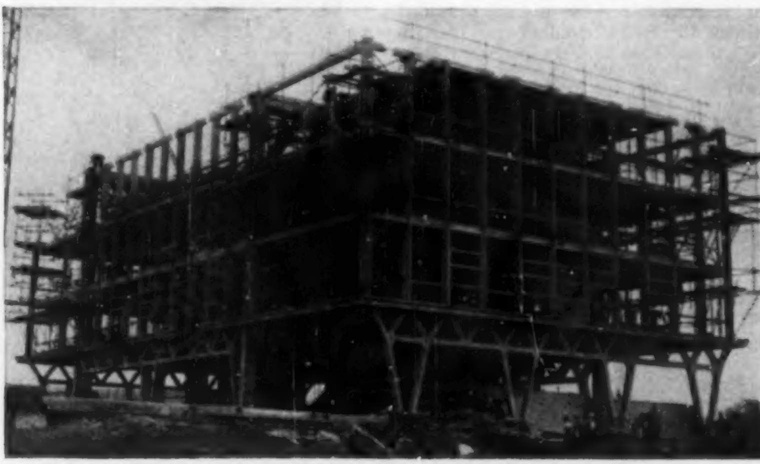


Fig. 1.—College at Derby.

Floors.

The average span of the floors is 26 ft.; they are 1 ft. thick and are constructed with prestressed precast inverted tee-beams and a 2-in. concrete topping which was cast on corrugated asbestos-cement sheets which serve as permanent shuttering. The beams are at 2 ft. centres at Ipswich, and 3 ft. 1 in. at Derby where the imposed load is smaller and the beams span between the external wall-frames and the central core or to the three-span main frames provided to resist the wind.

Precast Wall Frames.

Each wall-frame is one story high and comprises two vertical members and a lintel cast in one piece. Precast sills are fixed by bolts to the frames after erection. The lintels project lengthwise beyond the vertical members. At Ipswich the projecting ends of adjacent frames abut (*Fig.* 4). At Derby (*Fig.* 5) there is a gap between the ends which is closed, after erection of the frame, by a precast piece.

Steel tubes bolted to each vertical member and to the floor are provided as temporary inclined struts (*Fig.* 4) to keep the frames in position during erection; the lengths of the struts are adjusted by turnbuckles. Halved joints are provided in the vertical members at Ipswich, where the bases of the vertical members of the frames immediately above the first floor are bolted to the floor. A similar connection (*Fig.* 3) is provided at each floor at Derby, where the frames are 12 ft. high, and the vertical members are 1 ft. 3 in. by 7 in. and are rebated and splayed. The lintel is 10 ft. 10 in. long including two cantilevers each about 2 ft. 3 in. long and the overall size is 1 ft. by 1 ft. The lintel is ell-shape in order to provide supports for the floor beams. The sills are 4 in. deep by 9 in. wide and are provided between all vertical members; at Ipswich sills are provided in alternate bays only.

Fig. 3.—Connection at Bottom of Wall Frame.

Lattice Girders.

Lattice girders and V-shape columns of reinforced concrete are provided as the main supports along some of the edges of the first floor in all the buildings (*Figs.* 1, 2, 5, and 6).

The girders along the front of the

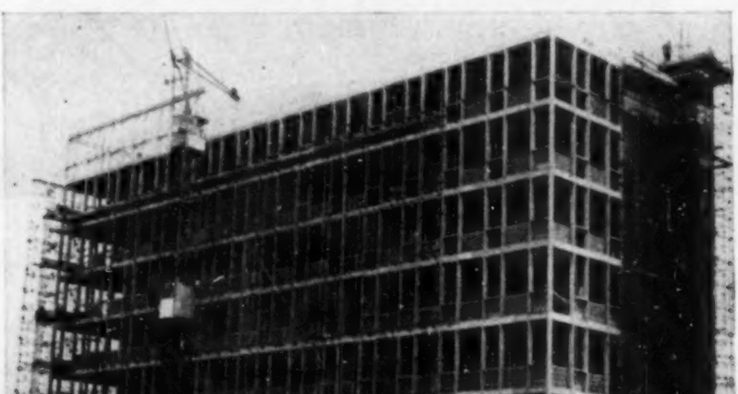


Fig. 2.—College at Ipswich.

building at Derby were precast in two parts each 31 ft. long, 4 ft. 10 in. deep, and 1 ft. 10 in. wide overall and each weighs about 8 tons. The bottom boom is horizontal and the top boom is notched to provide a support for the first-floor beams. Bars projected outwards from the top boom to connect to the gutter, which was cast in place. The girders were supported on staging during erection, and the connection between the two parts, the connections at the ends, and the connections to the inclined members forming the V-shaped columns, which were cast in place, were made by placing concrete around bars projecting from the members, so that a monolithic frame is formed. The lattice girders at the end walls were also precast but are shorter; therefore the inclined members of the V-shaped columns were cast monolithically with the girder to form a precast unit 18 ft. 6 in. by 18 ft. 6 in. overall and weighing about 4 tons.

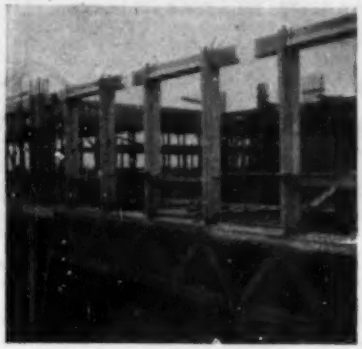


Fig. 5.—Wall Frames.

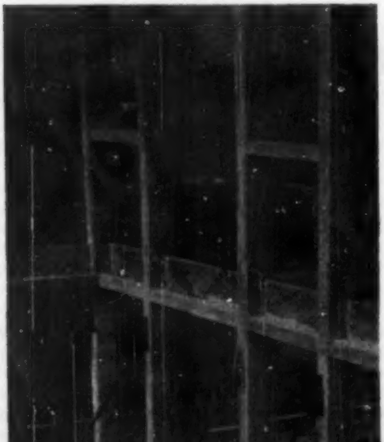


Fig. 4.—Precast Wall Frames.

The bottom booms of the precast lattice girders along one face of the larger of the teaching buildings at Ipswich college are not horizontal (*Fig.* 6) and contain ducts to receive prestressing wires. The V-shaped columns are generally at 20 ft. centres but are at 30 ft. centres at the main entrance, and were precast. The columns and lattice girders were erected and propped in position and the joints between the precast parts were grouted. When the grout had hardened, 0·276-in. wires in the bottom boom were tensioned to prestress the girders.

Assembly Hall.

The assembly hall to be erected at the Ipswich site will have a floor area of 3000 sq. ft. The roof of the tower over the stage will be of column-and-beam construction cast in place, but the roof over the auditorium will comprise 2½-in. reinforced concrete slabs cast in place to form a series of inverted vees, the distance between the apexes of which will be 10 ft. and the height 3 ft. The roof will rise 11 ft. from the eaves to the ridge, and will be supported by precast trifurcated columns.

The consulting engineers for the two colleges are Messrs. Felix J. Samuely & Partners. The architects for the Ipswich college are Messrs. Johns, Slater & Howard, the general contractors are Messrs. J. Gerrard & Sons, Ltd., the prestressed floor and roof units and the precast members of the roof of the

Fig. 6.—Columns and Lattice Girders.

assembly hall are supplied by Messrs. Saunders (Ipswich), Ltd., and the precast units for the main teaching buildings are supplied by Messrs. Cawood Wharton & Co., Ltd. The architects for the Derby college are Messrs. Grenfell Baines & Hargreaves, the general contractors are Messrs. Gee, Walker & Slater, Ltd., and the contractors for the precast concrete work are F.C. Construction Co., Ltd.

Walls Acting as Beams.

The details on page 33 are of the walls of the secondary modern school now in course of construction at Richmond, Yorkshire. The two-story building is about 300 ft. long and 75 ft. wide. Transverse reinforced concrete walls, which are at 25 ft. centres between the classrooms on the first floor, act as beams supporting the first floor and spanning 52 ft. 6 in. between columns 2 ft. 6 in. by 1 ft. and cantilevering about 10 ft. at each end. A large unobstructed area for the dining hall, assembly hall, and gymnasium is thereby provided on the ground floor.

The walls are about 10 ft. high and are designed to resist the lateral bending moments caused by the unbalanced loading on either side, tensile forces due to the suspension of the first floor, and as beams supporting the first floor and roof.

Doorways are provided through the walls, and a strip of glazing 20 ft. wide is provided along the middle of the roof. The greater depth of the wall at the glazed part of the roof compensates partly for the absence of the roof slab in this position and which would form part of the compression flange of the wall beam.

The walls were cast in lifts of 4 ft. for the entire length of 75 ft., and support the reinforced concrete roof and the first floor which is monolithic with, and suspended from, the walls. The first floor comprises prestressed precast beams 5 in. wide and 7 in. deep at 2 ft. 3 in. centres. The beams support wood-wool slabs 2 ft. square and 2 in. thick on which 2½ in. of concrete is cast in place. Each precast beam and the topping forms a tee-beam which is continuous over five bays. The beams at the edges of the floors are 1 ft. deep and about 2 ft. wide, and have a projecting nib. They were cast in place and support the glazing of the external walls, and provide a trimming to the ceiling-board which is nailed to timber fillets built in the underside of the ribs of the prestressed beams. Double walls are provided at the transverse expansion joints, which are 125 ft. apart.

Since the concrete of the transverse walls is exposed, horizontal vee-shape recesses are provided along the line of the construction joints and form a decorative feature in alignment with the heads of doors and black-boards.

The architect is Mr. D. Clarke Hall, the consulting engineer is Dr. K. Hajnal-K nyi, and the contractors are Messrs. F. Shepherd & Son, Ltd.

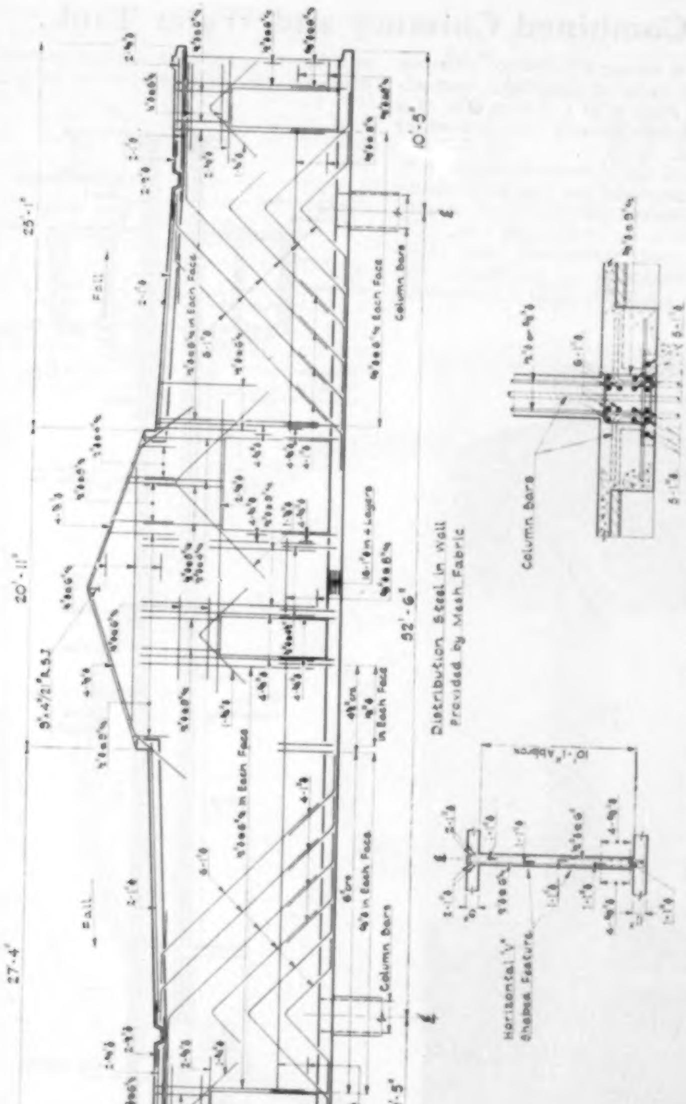


Fig. 1.—Details of a Typical Wall Acting as a Beam.

(See facing page.)

A Combined Chimney and Water Tank.

In *Fig.* 1 is shown a combined chimney and water tank at Aldridge, Staffordshire, and *Figs.* 2 to 4 give details of a similar structure recently built at a school at Stafford.

The top of the chimney is about 70 ft. above the base and the internal diameter of the reinforced concrete shaft is 3 ft. and the thickness of the wall 8 in. The shaft is lined throughout with acid-resistant brickwork 4½ in. thick. The uppermost 3 ft. of the shaft is constructed in concrete made with sulphate-resistant cement. The shaft projects 6 ft. 8 in.

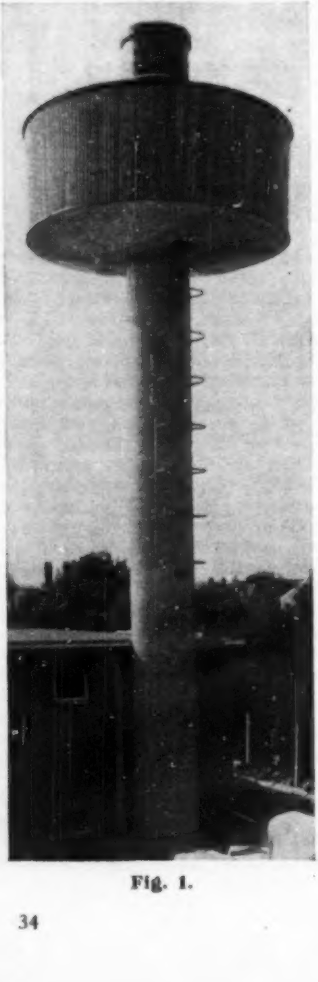


Fig. 1.

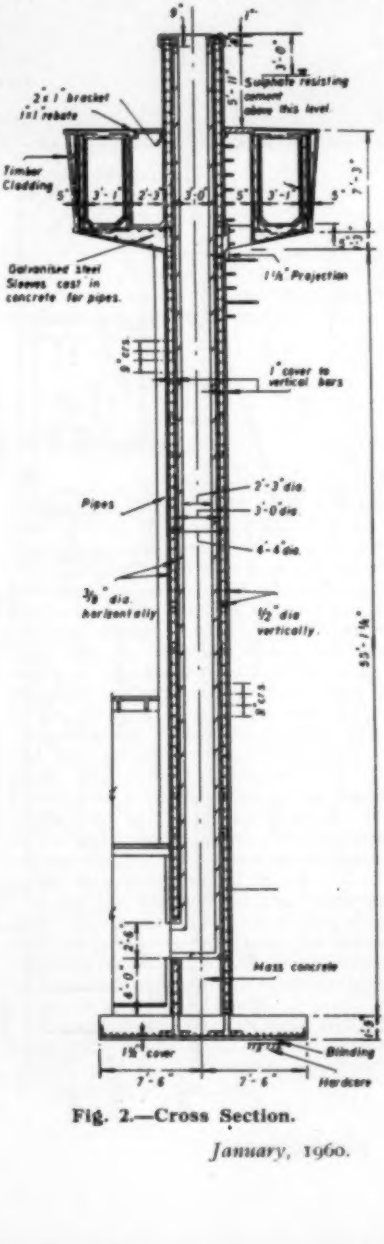


Fig. 2.—Cross Section.

above the cylindrical water tank, the bottom of which is about 55 ft. above the base of the chimney. The internal diameter of the tank is 15 ft. 10 in. and its capacity is 6500 gall. Because the wall is only 5 in. thick, the tank is lined with asphalt. The outside of the tank is faced with timber.

The foundation of the chimney is a slab 1 ft. 9 in. thick and 15 ft. square, reinforced with ⅝-in. twisted square bars at 6 in. centres in two directions.

The County Architect for Staffordshire is Mr. A. C. H. Stillman. The design was prepared by the British Reinforced Concrete Engineering Co., Ltd., who also supplied the reinforcement. The contractors for the structure described were Tideswell Brothers, Ltd.

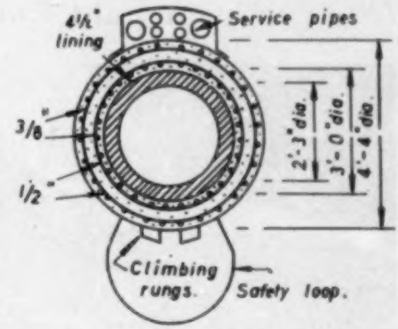


Fig. 4.—Sectional Plan of Chimney.

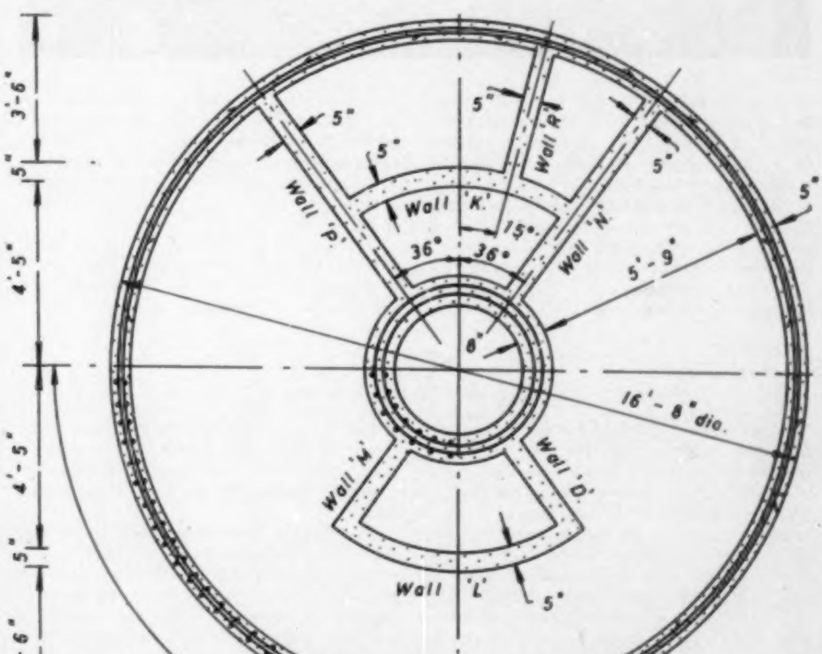
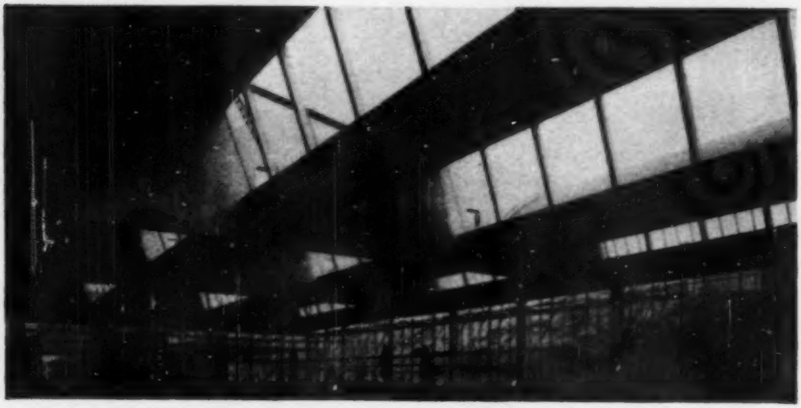


Fig. 3.—Sectional Plan of Chimney and Tank.

North-light Roof for a Factory.



THE north-light roof illustrated is at the factory at Redditch of Messrs. Herbert Terry & Sons, Ltd. It covers 57,600 sq. ft. and comprises twenty-four units of "shell" construction, each unit being 26 ft. 6 in. wide and 90 ft. long including a 20-ft. cantilever. The units are in groups of three and are carried on beams prestressed with post-tensioned cables by the P.S.C. method, each cable containing twelve 0·276-in. wires. Intermediate columns are provided at the junctions of the units only. The longitudinal valley beams accommodate the ducts from the air-conditioning plant, and all the beams are designed to enable heavy factory equipment to be suspended from them.

The architects are Messrs. Francis W. B. Yorke, Harper & Harvey. The design and construction of the reinforced concrete were by Truscon, Ltd.

Bridge with a Span of 500 ft.

TENDERS have been invited by the Ministry of Transport for the construction of a bridge to carry motorway M2 over the River Medway near Rochester. The deck will provide two carriageways, cycle tracks, and footpaths, and will be at a height of about 116 ft. above the river. There will be a central span of 500 ft. and side spans of 313 ft. each over the river and seventeen spans of from 100 ft. to 135 ft. in the viaducts forming the approaches.

For the main bridge, tenders may be submitted for two designs in steel and one in prestressed concrete. The concrete design comprises hollow reinforced concrete piers supporting longitudinal prestressed hollow girders over each side span and cantilevering 200 ft. over the central span; suspended girders 100 ft. long are to be supported on the ends of the cantilevers. The designs in steel comprise two hollow girders supporting a steel deck.

The piers of the viaducts are to be reinforced concrete frames supporting prestressed precast beams and a slab cast in place. The beams are designed as freely supported members carrying the dead load but acting as continuous members in conjunction with the slab to carry the live load.

The piers of the main bridge will be founded on hard chalk. The piers of the viaducts will be supported on piles or footings.

The consulting engineers are Messrs. Freeman, Fox & Partners.

Jetties for Oil Tankers on the Thames.

PRECAST BEAMS.

Two new jetties and adjacent dolphins (*Fig.* 1) recently completed at Thames Haven for London & Thames Haven Oil Wharves, Ltd., have a dredged depth of 45 ft. and can accommodate oil-tankers of 80,000 tons dead-weight. The positions of the jetties are such that advantage is taken of the velocity and direction of the currents at various depths at all stages of the tide without interfering with the use of two adjacent older jetties.

Jetties.

The jetties are constructed of precast reinforced concrete in bays each 30 ft. long, supported on piled trestles. The concrete deck was cast in place. The hollow square piles are of steel, on which reinforced concrete cross beams were cast and carry precast longitudinal reinforced concrete beams. The lengths of the jetties are 474 ft. and 774 ft.

The jetty-heads are structurally identical (*Fig.* 2), and comprise two reinforced concrete slabs, generally 3 ft. thick, supported on eighty vertical and sixty raking steel hollow square piles. The upper deck is about 7 ft. above mean high water at spring tides, and the lower deck is 8 ft. above mean low water at spring tides. Twelve piles are parallel to the axis of the jetty at a slope of 1 in 2½, twenty-four piles are at right-angles to the axis at a slope of 1 in 2½, and twenty-four similarly at a slope of 1 in 3. The lateral raking piles, which are at the ends of the structure, are 115 ft. long and support loads up to 95 tons. The average length of the vertical piles is 105 ft. and each pile supports a load of 80 tons. After welding, the piles were sand-blasted, primed, and painted with two coats of bituminous enamel. When the piles had been driven they were filled with concrete after removing any water in them.

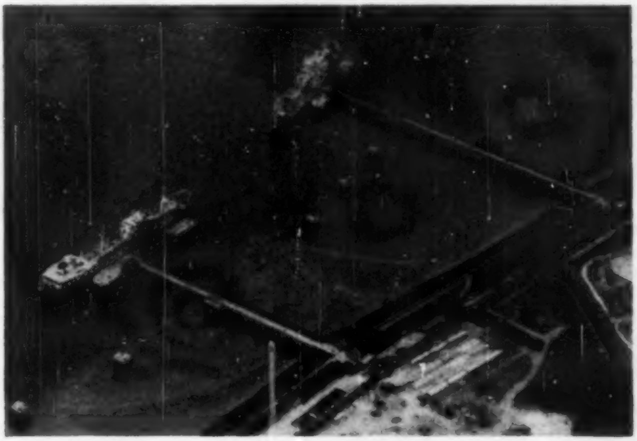


Fig. 1.

The soffit of the lower deck is 2 ft. 4 in. and 5 ft. above mean low water at neap and spring tides respectively, and the time available for erecting and removing the shuttering, even in calm weather, was therefore short. The upper deck was constructed first. The shuttering was supported on 1-ft. 6-in. by 7-in. steel joists spanning across the entire width of the jetty-head. The ends of the joists were supported on longitudinal joists, which were suspended from the outer vertical piles by high-tensile steel rods. When the concrete in the upper deck had hardened the pile bracing was removed, the soffit

shutters were lowered a convenient distance, and the shuttering was altered to suit the lower deck. The shuttering was then lowered at low tide to the level required for the lower deck and secured to the vertical piles to resist the action of the waves. When the lower deck had been cast the shutter was again lowered and the joists were pulled out transversely by a crane on a barge. The amount of concrete in each placing was 65 cu. yd. in the upper deck and 33 cu. yd. in the lower deck. The total quantity of concrete in the two jetty-heads is 6500 cu. yd. The concrete for the lower decks amounted to 3000 cu. yd., and was deposited through metal trunking passing through temporary holes in the upper deck.

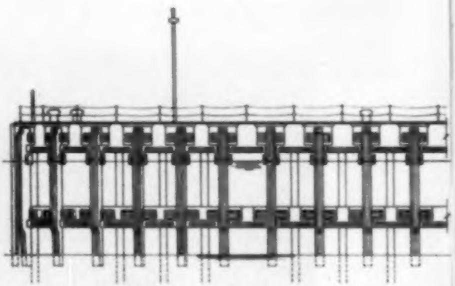
Fenders.

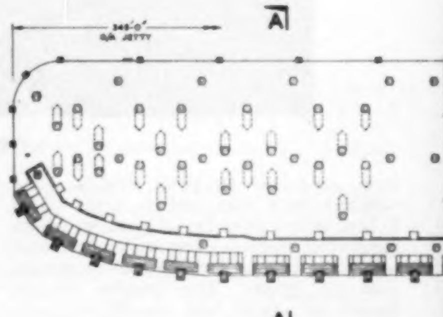
The fenders (*Fig.* 2) are of greenheart timbers 1 ft. 6 in. square faced with elm rubbing-strips. Each fender is supported laterally on a stout greenheart cross-head at the level of the top deck and at the lower deck. Two solid rubber cylinders, which when compressed absorb part of the energy, are provided between the cross-heads and the decks. The basis of the design is that the fenders should be capable of absorbing 2800 in.-tons of kinetic energy, this being a quarter of the energy of a vessel of 45,000 tons dead-weight (60,000 tons displacement) approaching the jetty at right-angles with a velocity of 1 ft. per second.

Dolphins.

Seven mooring dolphins are provided. Each dolphin, which comprises a concrete slab 21 ft. by 27 ft. and 5 ft. 6 in. thick supported on five vertical and eleven raking square hollow steel piles, is designed to withstand a pull on the bollards of 150 tons. The shuttering for the underside of the slab was suspended from the four vertical corner piles and was designed to support the weight of concrete in the first casting of one-third of the thickness of the slab. The side shutters were erected for the entire thickness of the slab. The concrete was deposited in three layers each containing 45 to 50 cu. yd. The first layer was allowed to harden for five days before the second was placed.

The working area, where the precast beams were made and the timber fenders were fabricated, was about 100 ft. behind the river wall. The concreting plant comprised two 21/14 mixers and equip-





PART PLAN OF LOWER DECK

ment for weigh-batching and storing loose cement, and was capable of an output of 17 to 20 cu. yd. per hour.

The preliminary investigations, design, and supervision of the work were made by Messrs. L. G. Mouchel & Partners. The contractors were Messrs. Peter Lind & Co., Ltd.

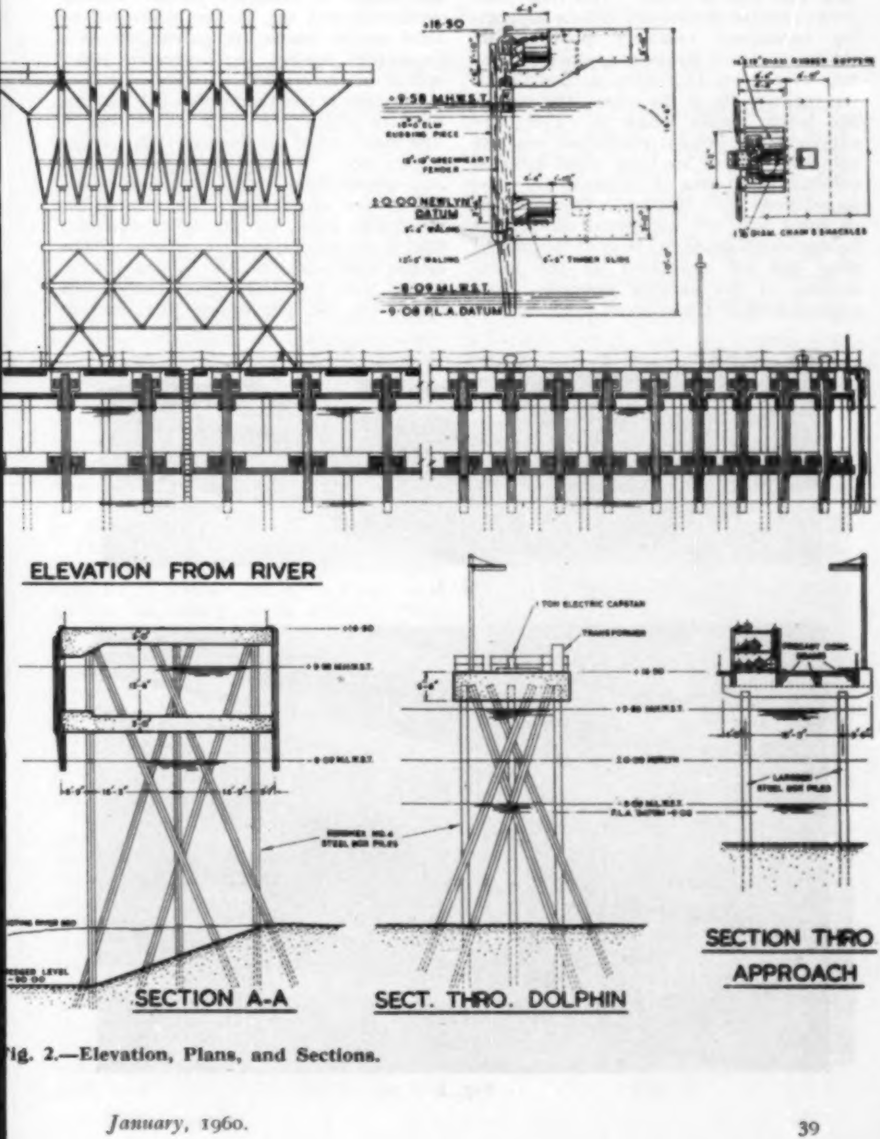


Fig. 2.—Elevation, Plans, and Sections.

A Low-cost Warehouse.

A NEW single-story warehouse (*Figs.* 1 and 2) at the chemical works of the Clayton Aniline Co., Ltd., Manchester, is 179 ft. long and 126 ft. wide. The roof comprises precast prestressed beams supporting reinforced concrete planks 2-in. thick. Natural lighting is provided by four roof-lights extending across almost the entire width of the roof. The beams, the height under which is 15 ft., are supported on precast reinforced concrete columns. There are only eight internal columns in an area of 22,800 sq. ft. The foundations are 7 ft. deep. The walls are of brick and are supported on precast beams which are at the level of the ground floor and are supported on the foundations of the exterior columns. The ground floor is a reinforced concrete slab with 1½-in. granolithic finish which is coloured and includes a hardening compound. The arrises of the columns and the edges of concrete ramps, loading platforms, and the like are protected by steel angles which are galvanised as a protection against the corrosive atmosphere of the works.

The time of construction was 5½ months, most of which were in a severe winter. The total cost of the work was 38*s.* 5*d.* per square foot. The cost of the ground floor and foundations, including excavation, was 5*s.* per square foot. The cost of the structural parts of the superstructure, that is the precast and prestressed beams in the walls and roof, the columns, eaves beams, 2-in. roof slabs, and canopy, was about 10*s.* 6*d.* per square foot, exclud-

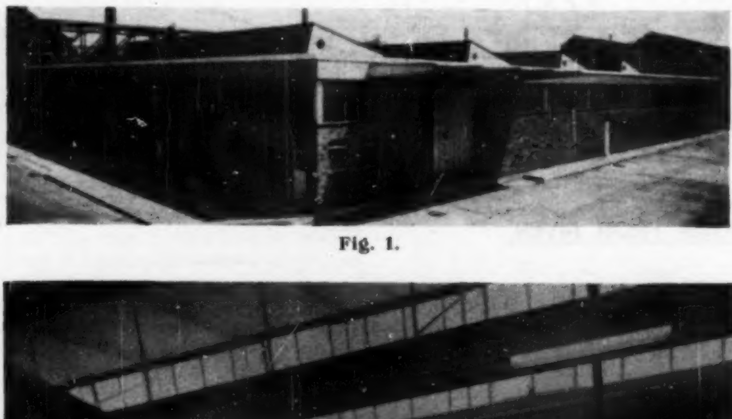


Fig. 1.

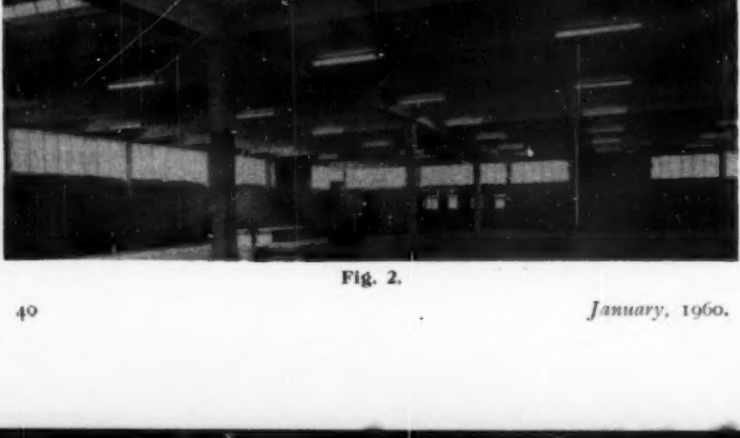


Fig. 2.

ing finishes, preliminary costs, and insurance.

The architect was Mr. S. Stothert, and the consulting engineers were Messrs. Taylor, Whalley & Spyra. The general contractors were Messrs. G. & W. Smith (Builders), Ltd., and the precast prestressed work was made and constructed by the Fram Reinforced Concrete Co., Ltd.

Fifteen-story Flats Built in Five Weeks.

THE first of six fifteen-story structures in Paisley, Renfrewshire, is shown in *Fig.* 1. The reinforced concrete columns, walls, and floors from ground to roof level were constructed in five weeks with the aid of sliding shutters which were raised at a rate of about 9 in. per hour by means of ninety jacks operated simultaneously from one position. The floor slabs are 5 in. thick and are supported by walls, which are generally 6 in. thick, and columns. The walls are faced externally with bricks supported on projecting concrete nibs at each floor level, and the nibs are covered with " brick tiles " 1¼ in. thick to give the appearance of a continuous brick wall. The imposed loads assumed in the design are 60 lb. per square foot on the floors and a pressure due to wind of 14·7 lb. per square foot on the walls.

The mixture of concrete was 250 lb. of cement to 641 lb. of washed sand and 1030 lb. of washed gravel to which 10·1 gallons of water were added ; this was calculated to be equivalent to a 1 : 1½ : 3 mixture. For concrete to be used in the sliding shutters a water-cement ratio of 0·55 was used and for other concrete the water-cement ratio was 0·37. The aggregate-cement ratio was 6·3. The average compressive strength of this concrete at 28 days was 6000 lb. per square inch.

To ensure a continuous supply of concrete during the raising of the shutters two 18/12 mixers were used, each having an hydraulically-powered shovel and weigh-batcher and a tank, which on the operation of a lever supplied sufficient water for a batch of concrete. The concrete was mixed at a rate of 10½ cu. yd. per hour, 1900 lb. of dry material being used in each batch. The mixers discharged into hoppers on bogies travelling on tracks, so that each mixer could supply either of two tower cranes which were used to elevate the concrete to the top of the shutters ; the cranes were 102 ft. high to the top of the tower and were capable of lifting 2½ tons at a distance of 30 ft. to a height of 176 ft. The shuttering for the floors was of plastic-faced plywood supported on telescopic centres. The period occupied in dismantling the shuttering and erecting it for the next structure was four weeks.

The buildings were designed by the Paisley Burgh Engineer, Mr. J. M. M'Gregor, O.B.E., and the general contractors are Messrs. Blackburn (Dumbarton), Ltd. The Prometo system of sliding shutters is used.

Fig. 1.

Reservoir and Water Tower at Ipswich.

THE structures illustrated in *Fig.* 1, which comprise principally a covered reservoir having a capacity of 4,000,000 gallons, a pump-house, and a water tower having a capacity of 125,000 gallons, are now nearing completion. The work was commenced in June 1958, and is for the Ipswich Borough Council.

Reservoir.

The reservoir has two compartments each 164 ft. long and 124 ft. wide. The height from the floor to the underside of the roof is 17 ft. 10 in. (*Fig.* 2). The floor and roof slope 1 ft. and the average depth of water is 16 ft. 11 in.; there is a freeboard of 5 in. at the overflow weirs. Baffle-walls are not provided, but circulation of the water is obtained by placing the inlet and outlet at opposite corners. The reservoir and water tower are on sand. The floor of the reservoir comprises a reinforced concrete slab 7 in. in thickness overlying a blinding layer of plain concrete 3 in. thick. The walls are designed as propped cantilevers and vary in thickness from 1 ft. 3 in. at the base to 8 in. at the top; the inner face is vertical. Excavation was simplified by forming the wall footings and column bases above the floor slab. The roof is a flat slab 7 in. thick with thickened panels and is supported on columns 1 ft. 2 in. square at 14 ft. centres.

No permanent contraction or expansion joints are provided but during construction gaps 3 ft. wide were left in the floor and walls at about 30 ft. centres. After not less than twenty-eight days, when most of the shrinking had taken place, the gaps were filled. The roof slab was constructed in alternate bays. At construction joints, the surface of the concrete placed first was hacked to expose the aggregate over the entire face of the joint. Water-bars were not provided.

On completion of the concrete work and before depositing the earth forming the embankments and cover, each compartment was filled with water for twenty-eight days. The tests were made during the dry summer of 1959, and the absence of the insulation, provided later by the earth, during the prolonged hot weather constituted a severe test. A few small leaks occurred, but by the end of the period of the test they stopped.

Fig. 1.

Water Tower.

The water tower is 84 ft. high and has a tank of 40 ft. internal diameter and 17 ft. 8 in. deep. The tank, the shape of which is octagonal on the outside, is lined with asphalt, and is supported on eight columns which taper from 2 ft. by 5 ft. at the foundation to 2 ft. by 4 ft. 2 in. at the underside of the tank. There are no intermediate braces. The central shaft is octagonal in cross section and encloses a reinforced concrete stair spanning

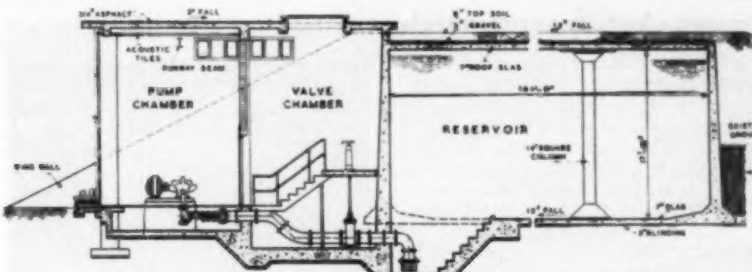


Fig. 2.—Section through Reservoir and Pump-house.

between opposite walls of the shaft leaving spaces from the ground to the underside of the tank on either side of the stair for pipes. Access to the roof is by means of a ladder in a small circular shaft in the centre of the tank. When it was tested the tank was quite watertight.

Water for the tank is supplied from the reservoir by three 24-h.p. centrifugal pumps each capable of delivering 500 gallons per minute; the pumps are automatically started in sequence as the level of the water in the tower falls.

Mr. J. B. Storey is the Borough Engineer & Surveyor and Water Engineer of Ipswich. The consulting engineers are Messrs. L. G. Mouchel & Partners. The contractors are Messrs. Howard Farrow, Ltd.

Retirement of Mr. R. V. Chate.

Mr. R. V. Chate, who has retired from the secretaryship of the Reinforced Concrete Association, will have the good wishes of a host of friends. Few men have so unobtrusively done so much to help the concrete industry. The Reinforced Concrete Association was started in the year 1931, when a few of the leading firms of reinforced concrete designers decided to form such a body with the primary object of furthering the interests of reinforced concrete as a sound and economical structural material, and in particular to negotiate with government departments and public authorities responsible for regulations and by-laws that sometimes hindered the use and development of reinforced concrete.

Mr. Chate, who had a long experience of design and construction in reinforced concrete, was invited to undertake the preliminary work of forming the Association, which was incorporated in 1932, and as its secretary he has since been the counsellor of a long succession of presidents and members of council. In the year 1945 he started the "Reinforced Concrete Review", in which periodical many valuable papers and other contributions have been published.

The Association has been fortunate in having as its executive officer a man who combines much wisdom and organising ability with sound technical knowledge, and whose unassuming manner has endeared him to all who know him. The extent of the ever-ready help he has unstintedly given to all who sought his advice, whether or not they were members of the Association, is known only to him. In his retirement Mr. Chate will be able to look back with pride at the progress made by the Association during the long period of his secretaryship, and in the knowledge that when he laid down the cares of office reinforced concrete had become accepted as the most important material for most types of structure and that most of the restrictive by-laws and regulations had been swept away. He was elected an Associate Member of the Institution of Civil Engineers in the year 1915 and a Member of the Institution of Structural Engineers in 1926.—H. L. C.

A Church with a Precast Roof.

A NEW Baptist Church (*Fig.* 1) at West Norwood, London, is 114 ft. wide, 95 ft. long, and 36 ft. high. There are also a side chapel and changing-rooms in a single-story building about 97 ft. by 27 ft. and 13 ft. high, a hall 102 ft. long, 40 ft. wide, and 22 ft. high, a store, and a basement. The church has a reinforced concrete frame and brick walls.

The roof of the church is at two levels (*Fig.* 2). The central part (*Fig.* 3) comprises precast curved ribs carrying 4-in. precast lightweight concrete slabs and supported on two longitudinal primary beams. The roofs over the sides of the church are at a lower level and comprise reinforced concrete slabs cast in place and spanning from the primary beams to eaves beams which are supported on the exterior columns.

The primary beams are 9 ft. deep and 1 ft. 9 in. wide and span 96 ft. 6 in. between the main columns at the ends of the church. They are designed to be freely supported and were cast in place in three parts, the middle part being cast after the ends. Parts of the shutters for the sides of the beams were omitted temporarily to allow the concrete to be placed in the lower part of the beam. Vibrators were used internally and externally to compact the concrete, and the bars in the beam were arranged (*Fig.* 4) so that vibrators could be inserted between them. For a length of about 1 ft. at each end the concrete was omitted temporarily, and into this space bars projected from the beam

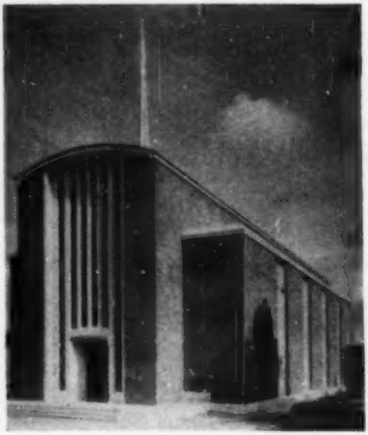


Fig. 1.

Fig. 2.

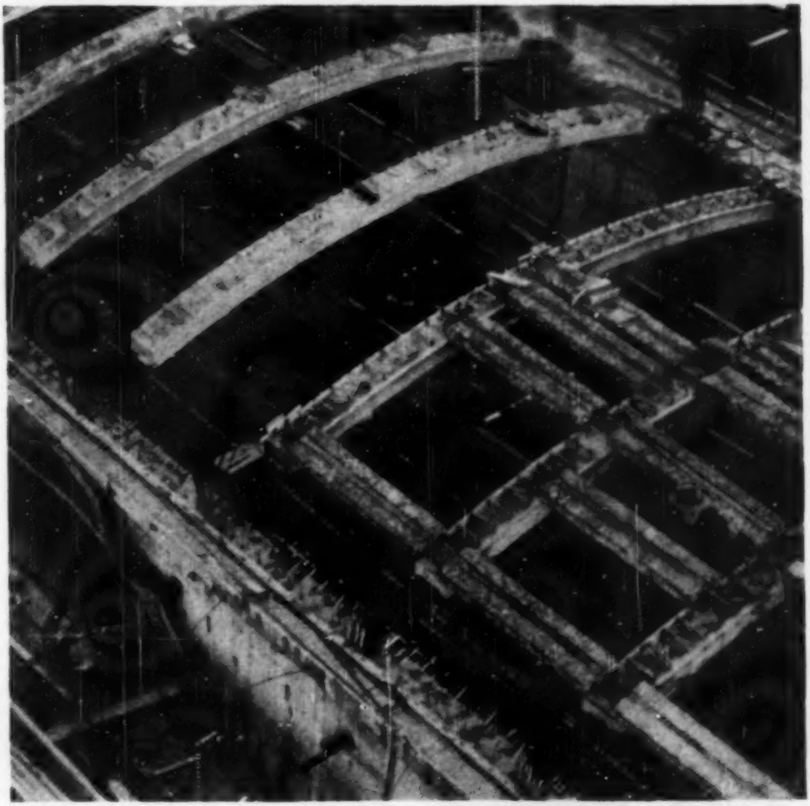


Fig. 3.

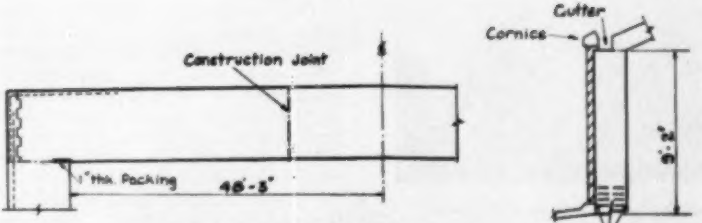


Fig. 4.—Elevation and Section of Part of Main Beam.

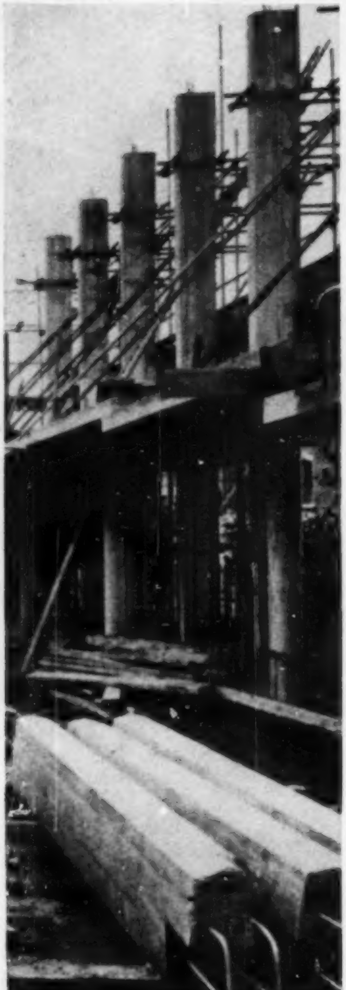


Fig. 5.—External Columns.

and from the column on which it rested. A strip of compressible material was placed between the column and the beam (*Fig.* 3) so that the joint between them was temporarily a hinge. When the concrete in the beams had hardened, the shuttering and props were removed and the beam was allowed to deflect. The props were replaced and the precast roof members were erected. The concrete was placed in the spaces at the ends of the beams and around the projecting steel, thereby forming a continuous joint with the column. In this way the risk of cracks due to the shrinking and deflection of the completed structure was greatly reduced.

The curved ribs are at intervals of 5 ft. 3¾ in., which conforms to the spacing of the fixings for the sheets of copper with which the roof is covered. The lower parts of the ribs were precast and bars project from them to form an anchorage to the compression flange formed by concrete cast in place between the ends of the precast roof slabs (*Fig.* 6). The slabs are of concrete containing 1¼ parts of rapid-hardening Portland cement, 1¼ parts of fine aggregate and 3¼ parts of lightweight aggregate in the form of nodules of fused pulverised-fuel ash; this concrete has a density of 90 lb. per cubic foot and a crushing strength of 1500 lb. per square inch.

The exterior columns (*Fig.* 5) were precast in two parts, the lower part extending up to the level of the balconies which are cantilevered reinforced concrete slabs cast in place. The cross section of both parts is approximately rectangular (*Fig.* 2). The main columns, which are ell-shape in cross section, form the ends of the side chapel and the end walls of the church. These columns are fixed at the bottom.

The foundations, which are designed for a pressure of 1 ton per square foot, comprise principally two reinforced concrete rafts, one under each end of the church, where the greatest concentrations of load from the main columns supporting the roof beams occur. The walls and intermediate columns are on strip footings which are assumed to distribute part of the weight directly to the ground and to transfer the remainder to the rafts between which the footings span. Under the raft at the western end of the church there is a slab of plain concrete 4 ft. 3 in. thick. The raft at the eastern end forms

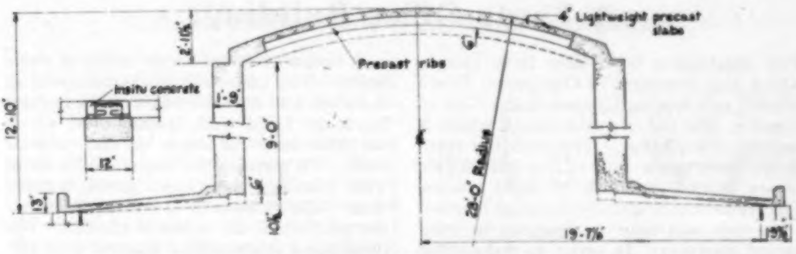


Fig. 6.—Cross Section through Roof and Detail of Precast Rib.

the floor of the boiler room, which is stiffened by two reinforced concrete beams.

The architects are Messrs. Ansell & Bailey, the consulting engineers are Messrs. Scott & Wilson, Kirkpatrick & Partners, and the contractors Messrs. H. Fairweather & Co., Ltd.

Prestressed Pipe-Bridges.

THE construction, near Barrow in Furness, of three prestressed bridges to carry two 21-in. water mains is nearing completion. The bridges, two of which are over tidal water, have spans of 75 ft., 65 ft., and 42 ft. and are about 7 ft. 6 in. wide, thereby allowing a walkway of about 2 ft. 6 in. between the saddles supporting the pipes. One of the bridges is illustrated in *Fig.* 1. The two larger bridges comprise four precast I-beams, each made up of five blocks which were assembled, jointed, and prestressed with cables at the site. The assembled beams are also prestressed transversely by short cables. The smaller bridge is of similar construction but the precast blocks are smaller.

Due to the low bearing capacity of the ground, each abutment is formed by three bored piles, the caps on which support the beams. Sliding joints, which are provided at each end of the two larger bridges and at one end of the smaller bridge, comprise copper plates one of which is attached by dowels to the underside of each beam and one on top of each pile-cap; a sheet of greased paper is placed between the pairs of plates. Anchor straps fastened to bolts built into the pile-caps are provided at each abutment to resist hydrostatic forces due to the change in direction of the pipes.

The design was prepared for Mr. H. C. Postlethwaite, Engineer of the Water Department of Barrow in Furness, by the British Reinforced Concrete Engineering Co., Ltd. The contractors are Messrs. Lane Brothers (Blackpool), Ltd.

Fig. 1.—Precast Beams in Position.

A Large Office Building.

THE illustration is of Bow Bells House, which has frontages to Cheapside, Bread Street, and Watling Street in the City of London, and the construction of which is nearing completion. The building comprises three parts—one of five stories, one of six stories, and one of eight stories. The frame was originally designed in steelwork but was later re-designed in reinforced concrete. In order to reduce the number of columns in the basement, part of which will be a car park and garage, some of the columns start at the ground floor which is designed as a distributing and stiffening slab and is 3 ft. thick. The basement is entered by means of a concrete ramp which is wide enough for two lines of traffic. A mechanical parking device will be installed. The foundation consists of bored piles of 2 ft. diameter and about 47 ft. long; the heads of the piles are 15 ft. below the level of the street.

The tank and lift-motor room are on the main roof and are in a penthouse, the roof of which cantilevers about 4 ft. but is apparently, but not actually, supported on circular precast concrete columns of 4 in. diameter and with an artificial stone finish. The underside of the balconies at the sixth and seventh floors of the highest block are faced with faience tiles, which are also used for some of the exterior walls. In general the walls will be faced with Portland stone and green ceramic tiles. The surrounds of the windows and the plinth will be of black granite. The three main stairs will be finished with terrazzo. The upper floors will be free of permanent partition walls. Cables for electrical floor heating will be embedded in the floor topping. A reinforced concrete cantilevered canopy faced with Portland stone is provided at the main entrance.

The time taken for the construction of the reinforced concrete frame and foundation piles was ten months, and the cost of this work was about £250,000. The architects are Messrs. Cotton, Ballard & Blow, and the consulting engineers are Messrs. W. V. Zinn & Associates. The contractors are Tersons, Ltd. The building is to be used as offices by Messrs. Balfour, Beatty & Co., Ltd.

A Winder Tower at a Colliery.

THE winder tower shown in *Fig.* 1 is at Rising Sun Colliery, Northumberland, and details of the design are given in *Figs.* 2 and 3.

The tower is of reinforced concrete and is 69 ft. long, 43 ft. 6 in. wide, and 110 ft. high to the level of the lower roof above which there is a penthouse containing the air-filters and fans. There is a ground floor, three upper floors, and a penthouse (*Fig.* 2). The two 750-h.p. friction-winders (W) are on the third floor (*Fig.* 3); the drums of the winding plant are at (D). There is also a 22-tons winch operating over sheaves in the penthouse on the roof for servicing the cages and ropes, and a 10-tons overhead travelling crane for servicing the winders and for lifting materials from the ground to any of the floors through the hatch (C). A well (L) for a lift with a capacity of ½ ton is also provided. On the second floor there are two electric generators and switchgear and four guide ropes for the counter-weights are attached to this floor; a 5-tons weight is suspended from each rope.

The main supports of the tower are four

Fig. 1.

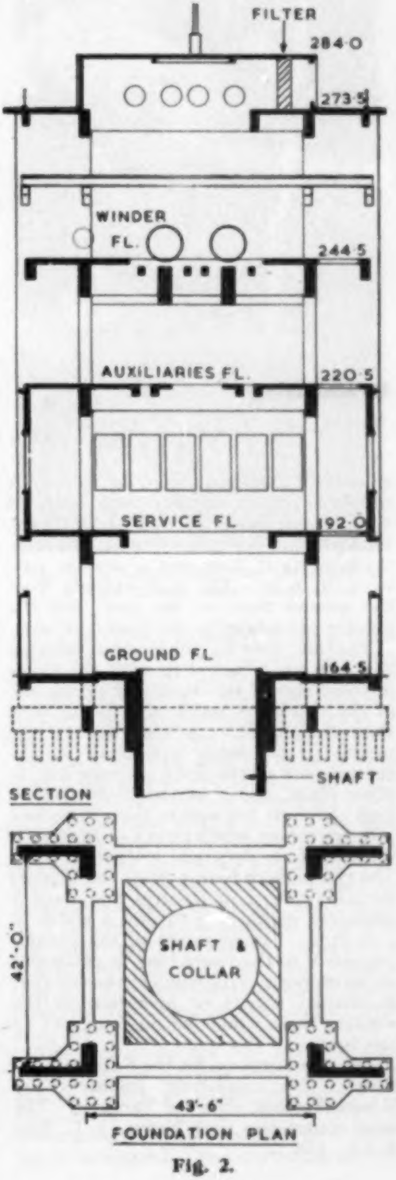


Fig. 2.

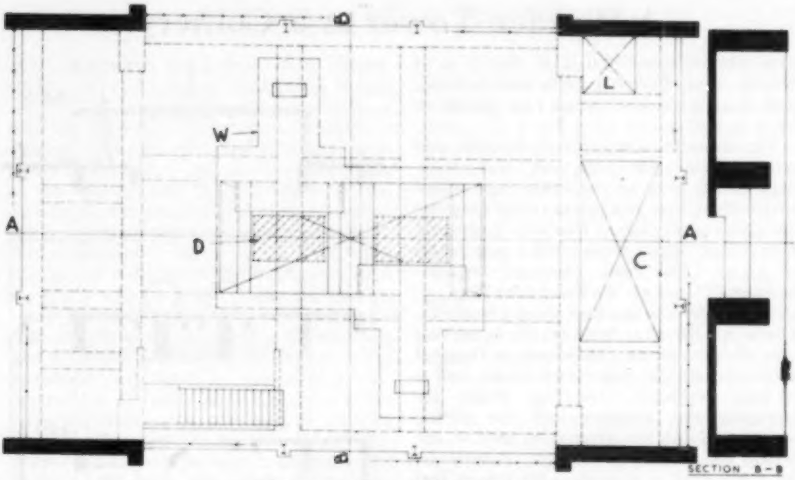


Fig. 3.—Plan of Third Floor.

ell-shaped columns each of which is supported on twenty-one bored piles of 1 ft. 5 in. diameter and 50 ft. long. Each column comprises a wall 1 ft. 6 in. thick by 14 ft. long and a pilaster 4 ft. by 2 ft. 6 in. The wall extends from the ground floor to the roof, but the pilaster extends up to the third floor only.

The third floor and the columns carrying this floor are designed to resist the forces imposed should the ropes supporting one of the two cages break, that is 220 tons from this cause, 124 tons due to the weight of the main equipment on the central area of the floor, 31 tons due to other plant, and a uniformly-distributed load of 100 lb. per square foot. The four primary beams, which form a 40-ft. square, are 8 ft. 6 in. deep and 2 ft. 6 in. wide. The two pairs of beams under the winders are at 12 ft. 3 in. centres and have a minimum depth of 7 ft. and a width of 2 ft. 6 in. Investigation of the natural frequency of the tower, and in particular of the supports of the winders, showed that at normal speeds of operation of the winders there would be no resonance in the beams.

The structures are for the National Coal Board. The consulting engineers are Messrs. Posford, Pavry & Partners. The main contractors were Messrs. J. L. Kier & Co., Ltd.

Lectures on Building.

The following lectures have been arranged by the Ministry of Works. Admission is free.

The Builder and his Contract. By N. P. Greig. Technical College, Denzil Road, London, N.W.10. January 19. 7.15 p.m.

Prevention of Accidents in the Building Industry. By J. A. Hayward. College of Building, Cauldon Place, Stoke-on-Trent. January 20. 7.15 p.m. Also at Lambeth Town Hall, London, S.W.2. January 26. 7.15 p.m.

Methods and Plant for Concrete. By A. B. Harman. New School of Engineering, University, Bristol. January 21. 6 p.m.

Work Study in the Building Industry. By R. Geary. Guildford Hotel, The Headrow, Leeds 1. January 22. 7.15 p.m.

Thermal Insulation of Buildings. By F. King. College for Further Education, Avenue Road, Grantham. January 26. 7.15 p.m.

Building and Civil Engineering Subjects (film show). Lord Nelson Hotel, Milford Haven. January 27. 7 p.m.

Practical Formwork Design and Construction. By J. G. Richardson. Technical College, Bell Street, Wakefield. January 28. 7.15 p.m.

A Large Reservoir in Kent.

NEW WALL SHUTTERS.

A SURFACE reservoir at Singlewell, near Gravesend, Kent, constructed for the Medway Water Board, has a total capacity of 4,000,000 gallons in two compartments, is 300 ft. long by 150 ft. wide and 17 ft. 6 in. high, and is constructed of reinforced concrete. The roof is supported on columns at intervals of 15 ft. 6 in. in both directions. Expansion joints with plastic water-bars are provided at intervals of about 31 ft. in the walls, floor, and roof. The floor was cast on building paper laid on a concrete sub-base. The foundations of the walls, and the walls to a height of 3 in. above floor level, were cast together and a metal water-bar is embedded in the construction joint at this level.

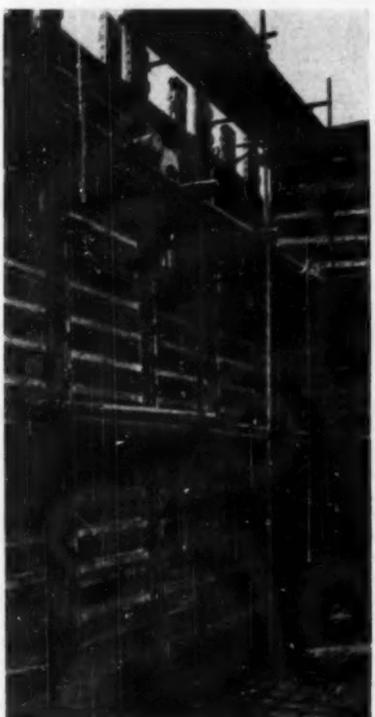


Fig. 1.—Internal Shutters.

Fig. 2.—Compacting the Concrete at a Gap in the Inner Shutters.

The concrete was mixed in the proportions of 1 : 1·6 : 3·2 with graded aggregate of ¾ in. maximum size and with a water-cement ratio of 0·56. The slump did not exceed 1 in. and the strength at 28 days was not less than 4500 lb. per square inch.

Parts of the side walls up to 30 ft. long were cast in one lift 17 ft. 3 in. high (*Fig.* 1) in a new type of shuttering which has hinged flaps 2 ft. 3 in. high through which the concrete is placed at different heights. The shuttering panels are 5 ft. high by 4 ft. wide and are bolted to vertical members which are 10 ft. high by 1 ft. wide and are stiffened by a tube and two mild-steel lattices which are welded to the back of the member (*Fig.* 2). The panels and vertical members comprise sheet-steel frames covered with ½-in. plywood.

The shuttering for the external face was moved, in sections about 31 ft. long and 18 ft. high, by a crane and supported in position by diagonal struts (*Fig.* 3) which are adjustable to enable the shuttering to be aligned. The shuttering for the

sloping inner face was fixed to the outer shuttering by spacers which passed through the wall and were variable in length. This shuttering was moved in sections 6 ft. wide and 18 ft. high, which were erected with gaps 4 ft. wide between them to provide access to the spacers. When these sections had been aligned, panels were bolted across the gaps up to a height of 6 ft.

Concrete was placed from a bottom-opening skip (*Fig.* 3) through hinged flaps in the outer shuttering; the flaps are hinged at the bottom (*Fig.* 4) and, when open, form chutes to direct the concrete to the interior. The concrete was compacted by means of poker vibrators passed through gaps in the inner shutters as shown in *Fig.* 2. When the concrete reached the bottom of the hinged flaps of the outer shutters, the flaps were closed and the gaps in the inner shuttering were covered to a level 6 ft. higher, to which level more concrete was placed. In this way sections of the walls up to 30 ft. long and 17 ft. 3 in. high were shuttered and cast and the shuttering removed within fifty-six hours.

Fig. 3.—View of External Shutter.

Fig. 4.—Hinged Flap through which the Concrete is Placed.

The reservoir was designed by the F.C. Construction Co., Ltd. The general contractors were Messrs. Thomas Lowe & Sons, Ltd. The shuttering is a new type produced by Kwikform, Ltd.

Prestressed Concrete.

THE Prestressed Concrete Development Group is arranging an exhibition of prestressed concrete at the Institution of Civil Engineers, Great George Street, London, S.W.1, from January 14 to 21. On January 14 the following papers will be read: "Prestressed Concrete Bridges in the British Isles", by Mr. A. Goldstein. "Prestressed Concrete Bridges Abroad", by Mr. A. J. Harris. "Research on Prestressed Concrete Bridges", by Mr. R. E. Rowe. "Bridges on the Maidstone By-pass", by Mr. H. Bowdler (County Surveyor of Kent) and Mr. F. M. Bowen. Tickets for admission are obtainable from the Secretary at 52 Grosvenor Gardens, London, S.W.1.

Grain Silo of 50,000 Tons Capacity.

A STRUCTURE (*Fig.* 1) being built at Meadowside, Glasgow, for the Clyde Navigation Trust provides storage for 50,000 tons of grain. The grain is brought in by conveyors through two gantries served by pneumatic unloading plants on the adjacent quay and is transferred to the bins by means of eight band-conveyors. The structure is of reinforced concrete, but the external faces of the walls of the bins are faced with brickwork. Other walls are of cavity construction comprising bricks and precast slabs with exposed granite aggregate.

The storage section is 90 ft. wide and 315 ft. long, and is divided into twenty-one rows of six bins each 15 ft. square, except in the end rows where there are twelve bins. A ten-story working-house 188 ft. high is provided at each end of the silos. Below the bins there are three stories providing accommodation for the chutes, automatic mobile weighers and, on the ground floor, three longitudinal railway tracks. Access for road vehicles is provided transversely. The main building is carried on a reinforced concrete raft 4 ft. thick bearing on boulder clay. The columns of the working-houses are supported on separate reinforced concrete footings each 10 ft. square.

Above the bins eight band-conveyors are arranged in two groups of four, one above the other. To provide space for the elevator platforms to the upper group of conveyors, the floor over the bins is at the unusual height of 20 ft., which, combined with the three stories below the bins, makes the height of the storage section 157 ft. Expansion joints are provided above and below the bins at about the third points and above the bins at the ends of the storage section. The bins are constructed monolithically. Services are in a two-story structure cantilevered from one face of the storage section at the levels of the first and second floors and extending the length of the middle third of this section.

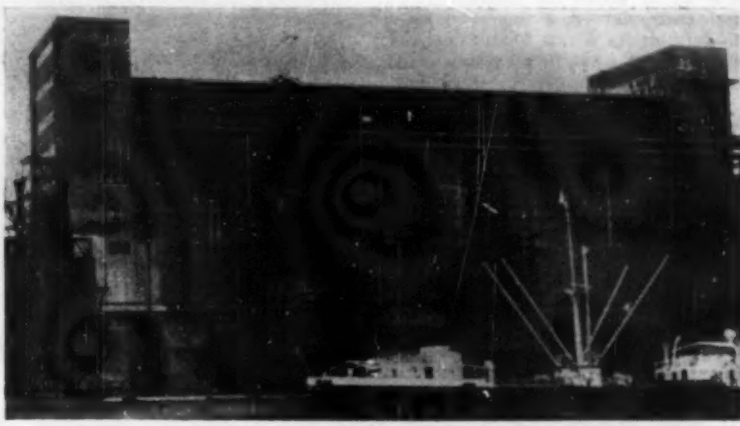


Fig. 1.

The bins were constructed in six separate parts by means of continuously moving timber forms suspended from steel yokes and operated by oil-jacks. The average time taken to construct the walls for each part was seven days. No two adjacent parts were erected successively, and the time between the completion of one part and start of the adjoining part was about 3½ weeks. The whole of the work built with moving forms was completed in less than six months. Because a facing of brick was to be applied to the walls of the bins, greater accuracy was necessary in the vertical and horizontal alignment of the forms; in the few cases where correction

was required this was achieved by the application of horizontal forces applied from a scaffold tower erected against the outer face of the wall.

The engineer of the Clyde Navigation Trust is Mr. A. Thomson. The consulting engineers are Messrs. L. G. Mouchel & Partners. The contractors for the civil engineering work, the cost of which is £725,000, are Messrs. Holst & Co., Ltd.

A Water Tower in Essex.

THE reinforced concrete water tower (*Fig.* 1) for the Southend Waterworks Company at Thundersley, Essex, has a capacity of 200,000 gallons. The tank is 56 ft. in diameter, and is supported on ten rectangular columns 2 ft. 3 in. by 2 ft. 4 in. in cross section and a central access shaft of 13 ft. diameter. The top water-level is 63 ft. above the ground.

Trial bores showed a fine sand for a depth suitable for a raft foundation and London clay at a depth of about 22 ft. Experience of this sand at a neighbouring site had shown its tendency to sponginess when it was exposed, so the structure is carried on sixty-five bored piles. One of the piles was tested to twice its working load by an hydraulic jack bearing against four adjacent piles; the settlement was 0·13 in., which was reduced to 0·07 in. when the load was removed.

The concrete in the foundation and central shaft was a 1 : 2 : 4 mixture, and in the columns and tank a 1 : 1½ : 3 mixture. The tank was designed in accordance with the Code of the Institution of Civil Engineers for Liquid-retaining Structures and was provided internally with a rubber lining. Twisted square reinforcement bars were used in the walls and floor so that any cracks that might occur would be widely distributed and of negligible width.

Access to the working platform below the tank is by a curved staircase fixed to one half of the circumference of the central shaft; each landing spans across the shaft as a chord of the shaft, leaving space on the other side of the shaft for the pipes for filling and cleaning the tank.

The lining is of natural rubber ¼ in. thick and was installed by the Dunlop Rubber Co., Ltd. Metalwork throughout (other than the Company's equipment) is of aluminium in order to reduce the risk of corrosion. The outer face of the concrete was washed with a slurry of cement and silver sand and rubbed with a carborundum disk.

Fig. 1.

Mr. Adam Hope is the Engineer of the Water Company. The consulting engineers were Messrs. Leslie Turner & Partners and the contractors Messrs. Holst & Co., Ltd. The piles were constructed by Holmpress Piles, Ltd. The cost of the work, excluding the pump and pipework, was about £29,300.

Precast Concrete in a Factory Building.

PRECAST members, most of which are prestressed, are being used in the construction of a two-story factory building in Battersea, London. The work now in progress comprises six parts, each about 76 ft. wide and 78 ft. long, which form the major part of the building, and expansion joints are formed to separate the parts. As is seen in *Figs.* 1 and 2 the roof is of a novel design.

The columns are of precast reinforced concrete, some to the full height of the building and some to first-floor level only; they are spaced at intervals of 39 ft. in one direction and 38 ft. in the other. The method of assembling the beams and columns, and a joint between two sections of the building, are shown in *Fig.* 3; at such a joint two main beams support the first floor. The first floor is formed of precast slabs 13 ft. long, 7 ft. 2 in. wide, and 6¾ in. thick which are supported on secondary beams 38 ft. long and containing 92 pre-tensioned wires of 0·2 in. diameter. When the slabs are in position concrete is placed around mild-steel reinforcement projecting from the secondary beam and in slots formed in the ends of the slabs (*Figs.* 3 and 6).

At the ends of each secondary beam on the underside are ⅜-in. mild-steel bearing-plates which rest on similar plates

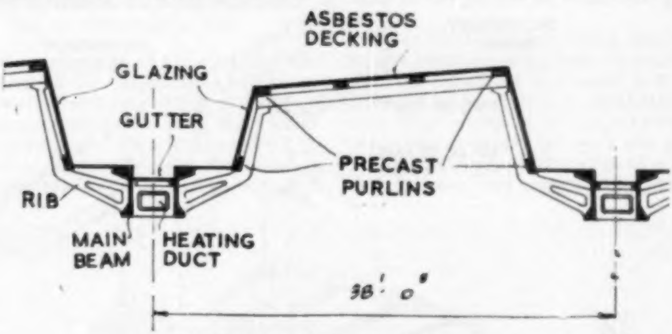


Fig. 1.—Cross Section of Roof.

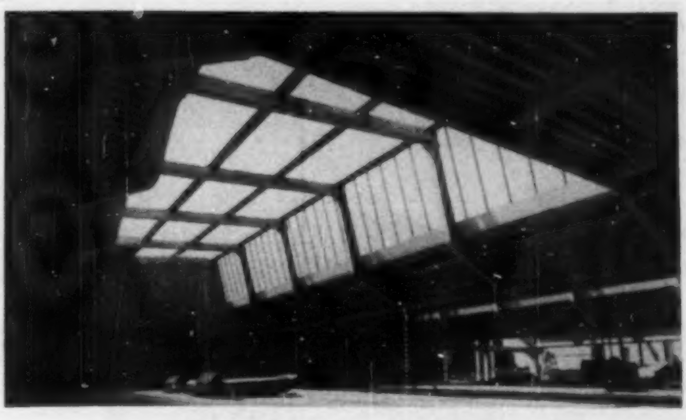


Fig. 2.—Precast Roof.

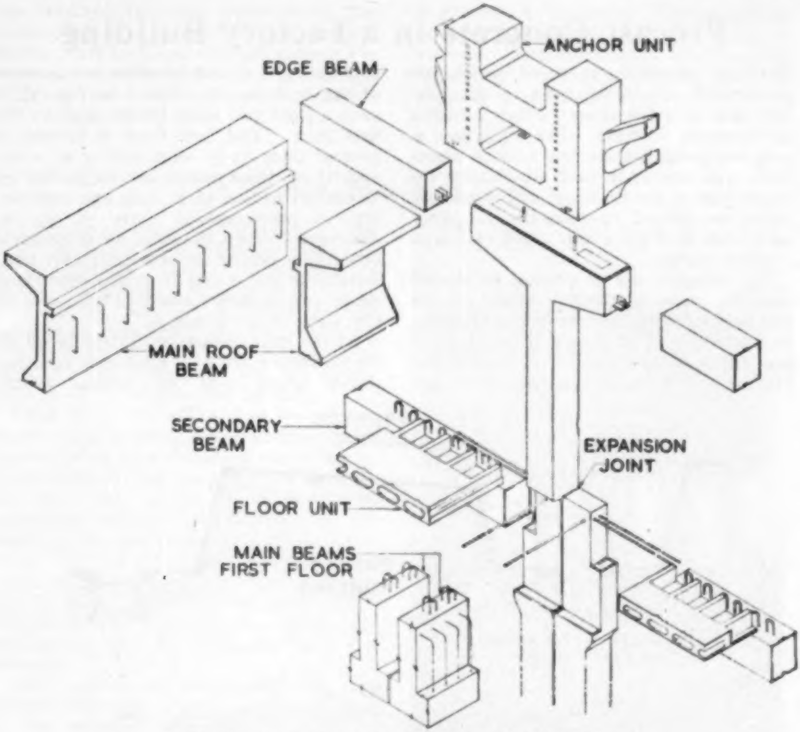


Fig. 3.—Arrangement of Columns and Beams.

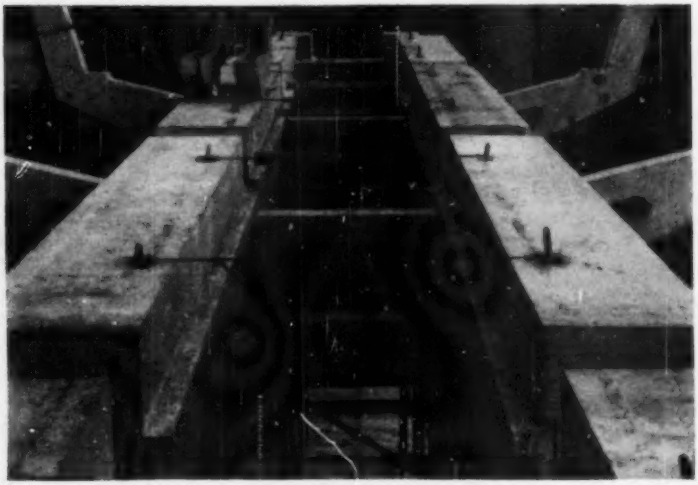


Fig. 4.—Parts of Main Beam in Position.

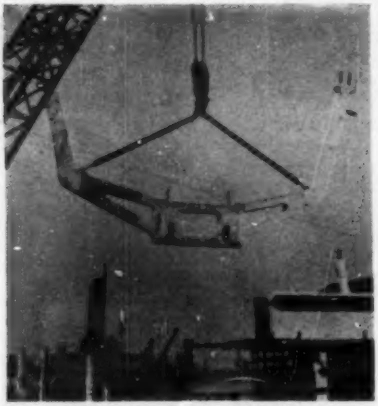


Fig. 5.—Erecting a Rib.

provided at intervals of 13 ft. along the lower flange of the main beams which are partly precast and partly cast in place.

The precast portion of each main beam is of inverted tee-section and about 38 ft. long and is prestressed with 220 wires of 0·2-in. diameter, each of which is tensioned with a force of 5200 lb. The top flange, which is level with the top of the floor, is cast in place around reinforcement projecting from the precast portion (*Fig.* 6). Concrete is also placed at the side of the beam between the lower flange and the soffit of the floor to provide added protection against fire.

Above the first floor the internal columns are discontinued and the roof is supported by precast beams about 78 ft. long bearing on the outer columns (*Fig.* 2). These beams are 4 ft. deep and are in two parts each with a top flange 20 in. wide and a bottom flange 10 in. wide (*Fig.* 3), and are precast in lengths of 11 ft. 10 in. In *Fig.* 4 one of these beams is shown assembled ready for concrete to be placed between the precast parts.

While the beams are being erected the precast parts are supported on scaffolding with the halves of the beam 3 ft. 7 in. apart. U-shaped ribs of reinforced concrete (*Figs.* 5 and 1) to carry the roof are erected transversely between the precast sections (*Fig.* 6). Fifty wires of 0·276 in. diameter are then placed along the inner

Fig. 6.—Construction of Floor and Roof.

face of each half of the beam and through steel plates in the ribs. The wires pass through two anchor-blocks provided at the ends of the beams (*Fig.* 7) and are fixed by P.S.C. 4-wire anchors. Concrete in the proportions 1 : 1 : 2½ with aggregate of ⅜ in. maximum size and rapid-hardening Portland cement, and having a maximum slump of ½ in., is placed in all the joints. When the concrete has hardened the wires are each tensioned with a force of 9900 lb., including an allowance of 16 per cent. for losses. The wires are then covered with concrete.

The cross section through the roof (*Fig.* 1) shows hollow prestressed purlins 14 in. wide and 6 in. deep supported by prestressed beams which rest on the arms of the ribs (*Fig.* 6) and are fixed in position by two bolts projecting vertically from the end of each arm. Before the main beams are prestressed weights are placed on a platform suspended from the centre of each beam (*Fig.* 2) to prevent it deflecting upwards while the roof is being erected.

Minor beams are connected to the columns by means of steel plates projecting from the lower part of the end of the beam and welded to steel tee-sections projecting from the column (*Fig.* 3). Reinforcement projecting from the upper portion of the end of the beam is threaded at the ends and connected by threaded sleeves to corresponding reinforcement in the column. When the beam has been aligned the connection is welded and concrete is cast around it to form a continuous joint between the beam and the column.

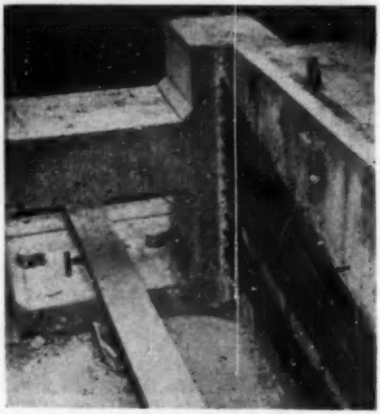


Fig. 7.—Cables at End of Roof Beam.

The architects are Messrs. L. D. Tomlinson & Partners, the general contractors are Messrs. W. H. Gaze & Sons, Ltd., and the design and erection of the concrete frames and floors are by Concrete, Ltd. Prestressing is by the P.S.C. MonoWire system.

Salaries in the U.S.A.

YOUNG men considering emigrating to the U.S.A. will be interested in an article in "Engineering News-Record" for June 4, 1959, which gives summaries of interviews with the heads of U.S.A. universities on the employment of graduates in civil engineering. It is stated that the demand for such men by state and local authorities and by industry is greater than ever before. The graduates experience no difficulty in finding employment, and are able to be selective in choosing an employer.

The starting salary for a graduate with a bachelor's degree is said to average 500 dollars a month. At the present rate of exchange this equals £2140 a year. Some idea of the value of such a salary in the U.S.A. may be had by comparing it with the rewards received in other walks of life. For example, the average basic wage of a bricklayer in New York is 4.25 dollars an hour; on the basis of 48 working weeks of 40 hours each a bricklayer's basic wage would be £2900 a year. In Great Britain the highest basic rate for a bricklayer is 4*s.* 9*d.* an hour, which amounts to £456 in a working year of 48 weeks of 44 hours each, and this may be compared with a starting salary of £700 or so a year offered in Great Britain to graduates in civil engineering. In the U.S.A. the graduate is paid less than a bricklayer's basic wage rate, whereas in Great Britain he is paid more than the bricklayer's basic rate—no doubt in the U.S.A., as here, the bricklayer's pay packet generally contains a good deal more than the basic rate agreed with the trade unions.

Eighteen-story Flats in London.

THE tall blocks of flats being erected at the Brandon Estate, Southwark, for the London County Council are now almost complete. There are six structures of eighteen stories (*Fig.* 1) and the construction, which is a combination of precast and cast-in-place concrete, was planned to enable the buildings to be erected without an external scaffold (*Fig.* 4); climbing cranes were used inside the buildings and tower cranes outside. A cross section is shown in *Fig.* 2, and plans of a typical floor and the foundations are shown in *Fig.* 3.

Foundations.

The ground consists of 4 ft. of filling overlying ballast and sand from 7 ft. to 24 ft. thick below which is London clay of a silty nature. The foundations comprise 180 bored concrete piles 20 ft. to 50 ft. long. The normal loads on the piles are 30 or 40 tons, which is increased by not more than 25 per cent. due to dead and imposed loading combined with the effects of wind. The lengths of the piles were determined to ensure uniform settlement of the pile caps.

Superstructure.

The loads are carried by concrete walls and columns cast in place. On the east

Fig. 1.

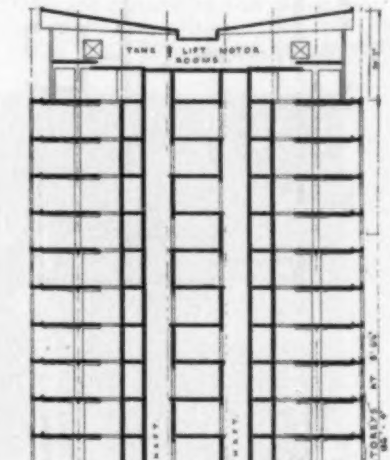


Fig. 2.—Cross Section.

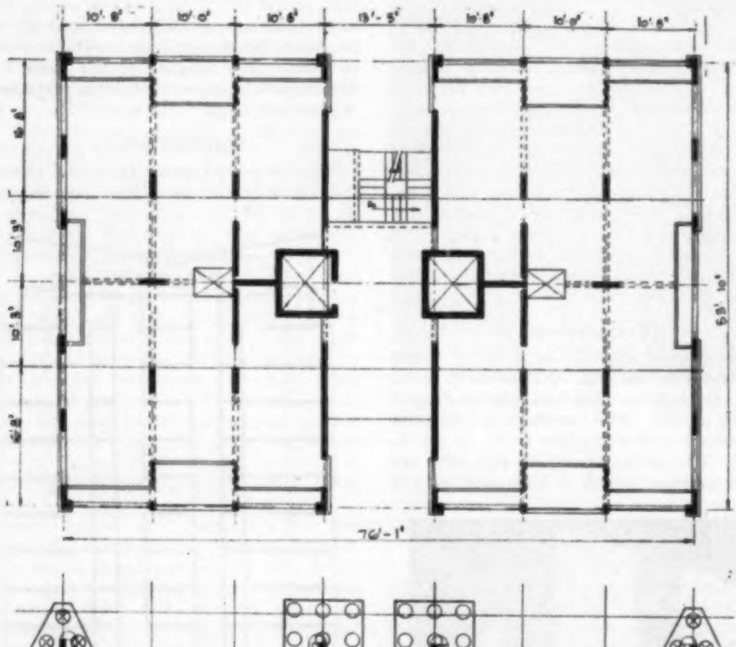


Fig. 3.—Plans of Upper Floor and Foundation.

and west elevations precast columns extend through four stories. At each fourth floor the columns are supported on a deep fascia beam, which is an architectural feature and is also of structural importance.

Wind forces are resisted by the concrete walls around the lift shafts and entrance lobbies acting as vertical cantilevers extending from the foundations. The structures on each side of the lobby act separately. The floor slabs are designed as horizontal beams transmitting the forces due to the wind to the walls. The dead loads on the walls are such that under the most adverse conditions of wind the tensile stresses in the walls are very small.

The floor slabs are 4 in. thick and span 10 ft. and were cast in place. They are supported on beams 6 in. wide, which coincide in position with the partitions between the rooms. Most internal columns and walls are 6 in. thick; the flanges of the columns also are in line with the partitions.

Precast slabs are used for the external walls and are supported on precast beams bolted to the columns (*Fig.* 5). The slabs, which have an exposed-aggregate

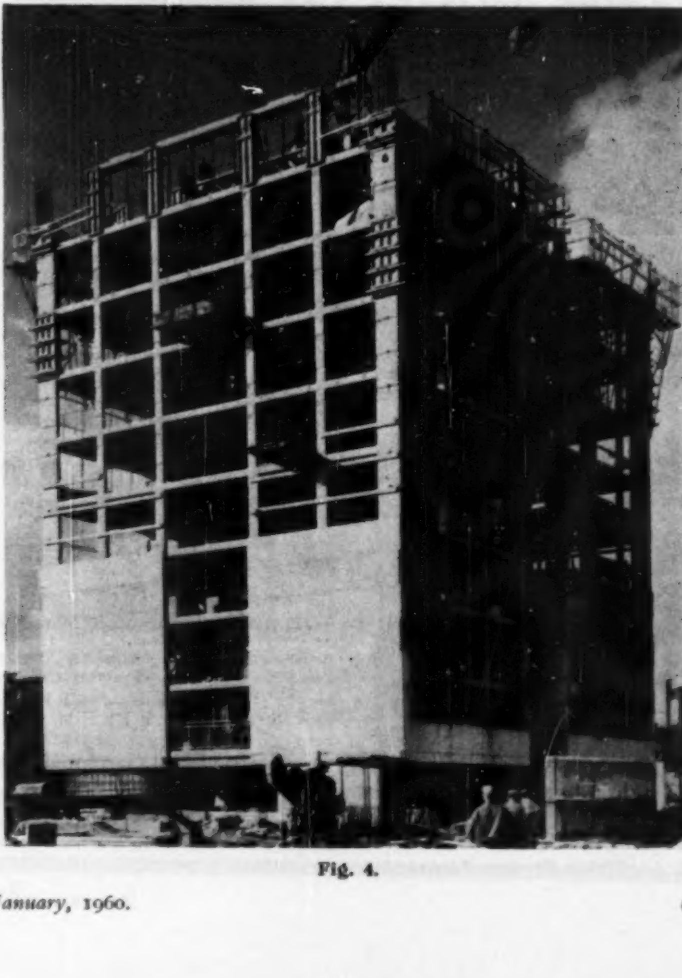


Fig. 4.

finish of white calcined flint, are designed to allow all fixing to take place from inside the building (*Fig.* 6). Sufficient clearance is allowed between the precast beams and the columns and also between the beams and the slabs to enable the beams and slabs to be fixed in their correct positions even if the members cast in place are slightly out of line or level. The joints are waterproofed by a plastic sealing compound, which was applied from a suspended cradle. Should the seal fail, or should condensation occur on the back of the slabs, a means of drainage is incorporated at the back of the slabs and moisture is discharged into a gutter at the bottom of the wall.

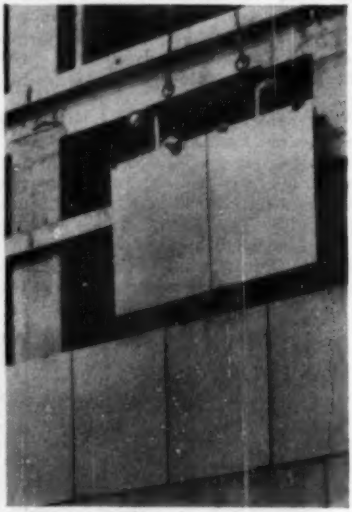


Fig. 6.—Fixing Wall Slabs.

Design Data.

The buildings were designed before the London County Council had accepted the recommendations of B.S. Code No. 114 (1957), and the design is therefore in accordance with the earlier by-laws of the Council. The concrete in the columns and walls in the lower stories is a 1 : 1 : 2 mixture, and the direct compressive stress permissible is 1140 lb. per square inch. Elsewhere the concrete is a 1 : 2 : 4 mixture and the compressive stress permissible in bending is 1000 lb. per square inch. These stresses were increased by 10 per cent. for vibrated concrete.

Reinforcement in compression and stirrups are of mild steel, but bars in tension are cold-twisted square bars the tensile stress permissible in which is 30,000 lb. per square inch. There are in each structure about 46 tons of high-tensile steel and 64 tons of mild steel.

The total cost of each structure, including the foundation, was about £58,000, of which about one-fifth was the cost of the foundation. The rate of erection was about one story in five working days. The buildings were designed by Mr. Hubert Bennett, Architect of the London County Council, and Mr. Whitfield Lewis, principal Housing Architect of the Council. The consulting engineers are Messrs. Felix J. Samuely & Partners. Professor A. W. Skempton advised on the foundations. The contractors are Messrs. Wates, Ltd.

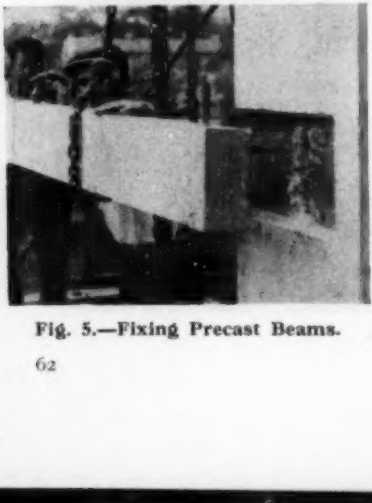


Fig. 5.—Fixing Precast Beams.

Grouting Prestressed Concrete.

A SYMPOSIUM on Grouting Prestressed Concrete will be held under the auspices of the Fédération Internationale de la Précontrainte and R.I.L.E.M. in Trondheim, Norway, from June 20 to 22, 1960. There will also be an exhibition of grouting equipment. Contributions for presentation at the symposium should be sent to Professor I. Lyse, Norges Tekniske Hogskole, Trondheim, Norway, by March 15, 1960.

Prestressed Roof Trusses.

THE curved roof of the assembly hall at West Derby Technical School, Liverpool, is shown on the right of *Fig.* 1. On the left is a small pyramidal roof which is raised to provide a clerestory.

The assembly hall is 79 ft. long, 60 ft. wide, and 18 ft. high to the eaves. Bowstring trusses are provided across the hall at 10 ft. centres and support prestressed tee-shaped purlins at 4 ft. centres with a covering of strawboard and bituminous felt. At one gable intermediate columns are provided and at the other gable a stronger truss was constructed with hangers to support a roof at a lower level.

The trusses have 9-in. by 8-in. rafters cast in place and prestressed ties comprising 5-in. by 4-in. precast members 3 ft. long with a hole on the longitudinal axis through which were passed $\frac{7}{8}$-in. high-tensile steel-alloy bars. A 1 : 1 cement-sand mortar was placed between the units, and when this had hardened sufficiently a pull of 18·3 tons was applied to each bar, which extended 2·73 in. The bars were then pressure-grouted in the concrete members and the ends of the bars cut off. The anchor-plates, which are 6 in. by 7 in. and $1\frac{1}{8}$ in. thick, project from each side of the tie into the concrete of the arch to transmit the thrust from the arch to the tensioned bar (*Fig.* 2). Each tie is suspended from the arch by two $\frac{1}{2}$-in. mild-steel bars encased in 4-in.-square precast units.

When the ties were in position the arch was cast, except for a length of 2 ft. at the middle which was cast fourteen days later to allow for shrinking. After fixing the purlins in position and when the concrete had sufficiently matured the shuttering was removed.

The gable arches are similar to the internal arches except that the ties were cast in place and prestressed with two $\frac{7}{8}$-in. tensioned bars in each and the rafters are 18 in. deep.

The roof of the kitchen consists of a

Fig. 1.

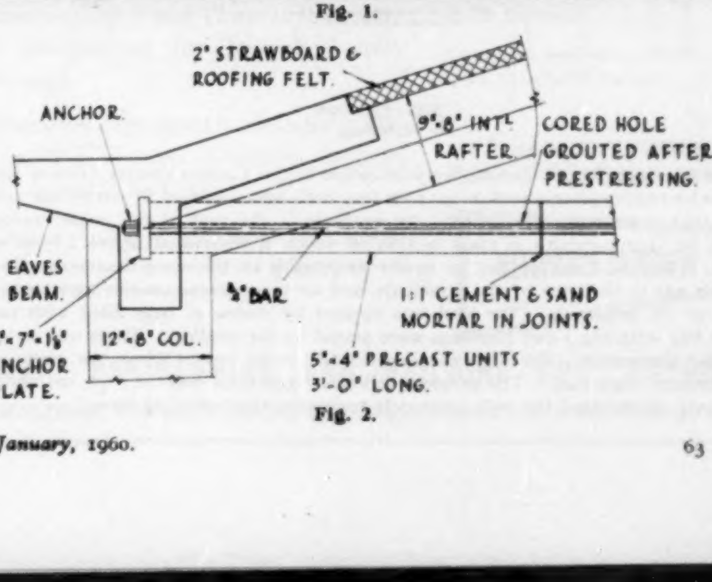


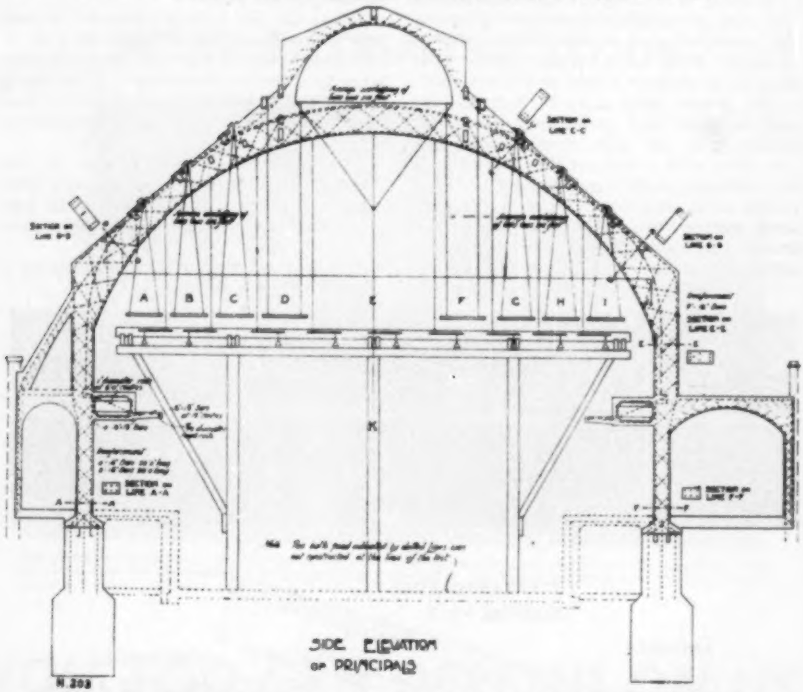
Fig. 2.

pyramid 19 ft. by 22 ft. on plan formed by four triangular slabs 4 in. thick supported at the corners by columns and stiffened along the edges with horizontal beams 2 ft. 4 in. wide.

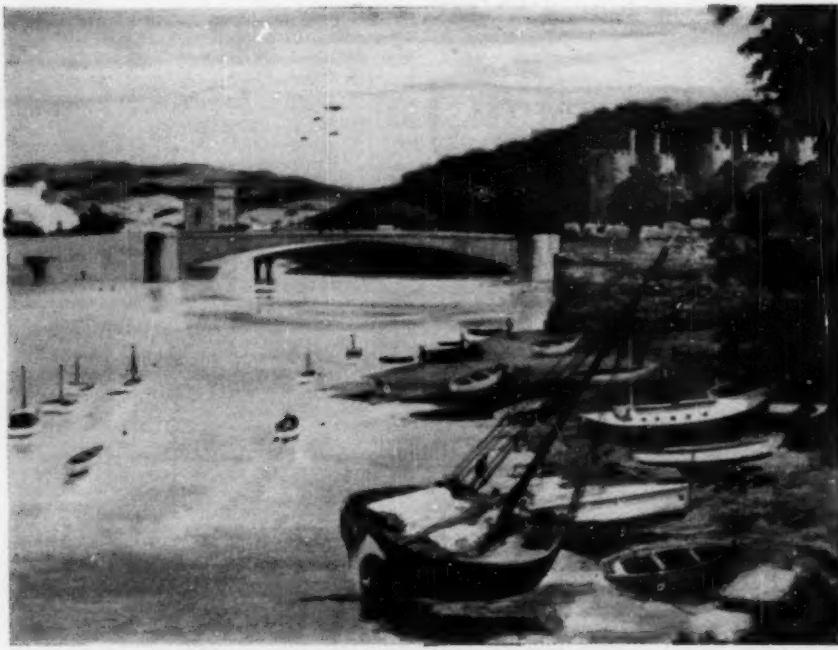
The roof was designed by the method described by Mr. A. J. Ashdown in " The Design of Prismatic Structures ". The architects are Messrs. Harold E. Davies & Son in collaboration with Dr. R. Bradbury, the City Architect. The main contractors were Messrs. R. J. Barton & Son, Ltd., and the precast members were made by Truscon, Ltd., who were the sub-contractors for the design and construction of the reinforced concrete work.

FIFTY YEARS AGO.

From " CONCRETE AND CONSTRUCTIONAL ENGINEERING ", January, 1910.



Test on Arch Roof.—Before the publication of the London County Council Regulations for reinforced concrete in 1915 loading tests were common to satisfy the authorities that a structure was sound. An example is the roof of the public swimming bath at Hammersmith, a cross section of which is illustrated above. It is stated that, " The load was applied as nearly as possible in the same manner as the load which was to be borne by the principals, and for this purpose cradles were slung from two of the principals. The load was applied by means of bags filled with ballast, each bag weighing 1 cwt (the bags were placed on the cradles). There was no spread at the abutments. The roof was superloaded to an extent of 50 per cent. of the permanent dead load. The greatest deflection measured was only $\frac{1}{25}$ in., and after removal of the load the roof principals recovered their original form."



CONWAY BRIDGE

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London Heliport.

A LANDING platform for helicopters was brought into use recently in London. The platform extends about 110 ft. from the south bank of the River Thames at Battersea, and comprises a concrete slab 1 ft. 9 in. thick without beams but supported directly on piles at 10 ft. centres in one direction and 12 ft. in the other. Owing to the difficulty of obtaining piles of one type in time, four different types were used, namely, reinforced concrete, prestressed concrete and two types of steel piles. The piles are about 45 ft. long. Raking piles are provided to resist lateral forces. The bed of the river comprises a thin layer of mud overlying a stratum of compact ballast about 13 ft. thick. The piles were driven through the ballast into the clay below by a diesel-hammer supported on the head of the pile. The piles and hammer were placed in position by a crane on the river bank, and were kept in

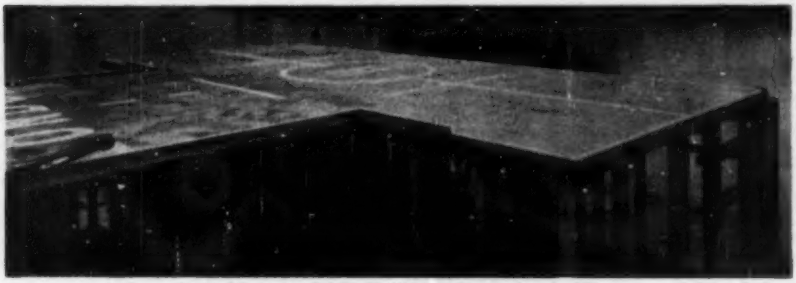

Fig. 1.—The Landing Platform.

Fig. 2.—Driving the Piles.

alignment while being driven through the hard ballast by a temporary steel structure (*Fig.* 2). Should it be necessary to raise the river wall in the future, provision has been made for the part of the platform at the wall to be cut away and replaced by a ramp. The architect is Mr. C. Oboussier, the consulting engineers Messrs. Lewis & Duvivier, and the contractors Messrs. W. & C. French, Ltd.

Stair with a Curved Roof.

THE stair illustrated in *Fig.* 1 is at the entrance to the extensions recently completed at Sunderland Technical College. The new works include classrooms, assembly hall, and workshops. The stair is at the entrance to the assembly hall, which is on the first floor. The structure of the hall comprises upper rectangular reinforced concrete frames 29 ft. high, the columns of which are 10 in. thick and taper from 30 in. wide at the top to 10 in. at the level of the first floor, where there is a hinged joint at the head of each of the columns in the lower story.

The entrance is between two of the upper frames. The external stair, which extends from the first floor to the ground, is covered by a curved concrete roof supported at one end on a beam between the two frames. The thickness of the roof is 2½ in. An intermediate support, beyond which the roof cantilevers 8 ft., is provided by two reinforced concrete raking struts which extend to the bases of the columns of the bottom story of the building.

The stair comprises precast concrete treads and a precast intermediate landing carried on two cranked stringer beams, which are supported intermediately by a beam between the raking struts.

The Borough Architect of Sunderland is Mr. H. C. Bishop. The reinforced concrete work was designed by the British Reinforced Concrete Engineering Co., Ltd., and the contractors were Messrs. Geo. H. Plemper Ltd.

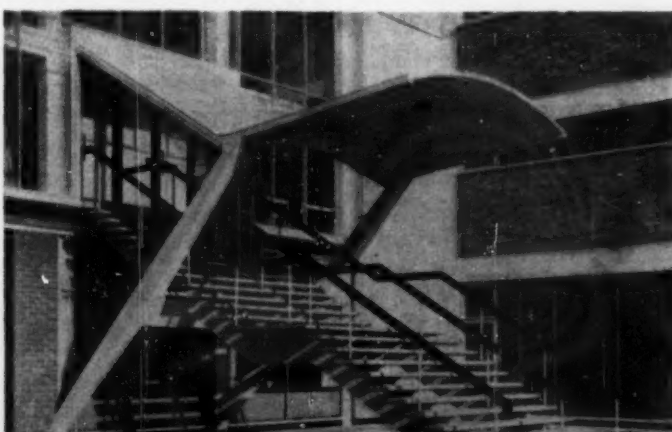

Fig. 1.

Prestressed Concrete Underpinning Beam.

THE entrance to a garage on the ground floor of an extension to a building occupied by Messrs. W. H. Smith & Son, Ltd., in Fetter Lane, London, is through the original building. To provide access of sufficient width it was necessary to remove a cast-iron column of 10-in. diameter supporting a corner of the older building, which comprises four stories and has load-bearing brick walls carried at the level of the first floor on steel girders spanning between cast-iron columns.

The load on the cast-iron column which was removed was transferred to a new beam spanning 24 ft. from a bracket on one of the columns of the new building to a new column provided to support the end of the beam (*Fig.* 1). The total load is 65 tons, and is applied at one of the third-points of the span of the beam, the depth of which was restricted to 2 ft. in order to provide sufficient height for the vehicles using the garage.

The depth of 2 ft. made a reinforced concrete beam impracticable, and a prestressed concrete beam was provided. The tendency of such a beam to camber upwards due to the prestressing force would counteract the downward deflection due to the load and thus greatly reduce any possible movement of the old load-bearing brickwork. To reduce the effect of possible settlement of the new foundation, the prestressed beam was not cast until the column of the new building was carrying most of its dead load.

A Freyssi flat-jack was provided at the top of the new column so that adjustment could be made for excessive deflection or settlement. A fire-resistance of two hours was required, and consequently not less than 2½ in. cover of concrete was necessary over the prestressing cables. At the plane of greatest bending moment on the beam, the head of the cast-iron column restricted the width of the beam in which prestressing cables could be placed. Consequently it was necessary for the beam to be 3 ft. 6 in. wide.

B.B.R.V. cables are used in order to reduce the number of cables and ensure that they should have the greatest possible eccentricity. Four cables tensioned to 93 tons in 2¼-in. diameter metal sheaths and two cables tensioned to 55 tons in 1⅞-in. sheaths are provided.

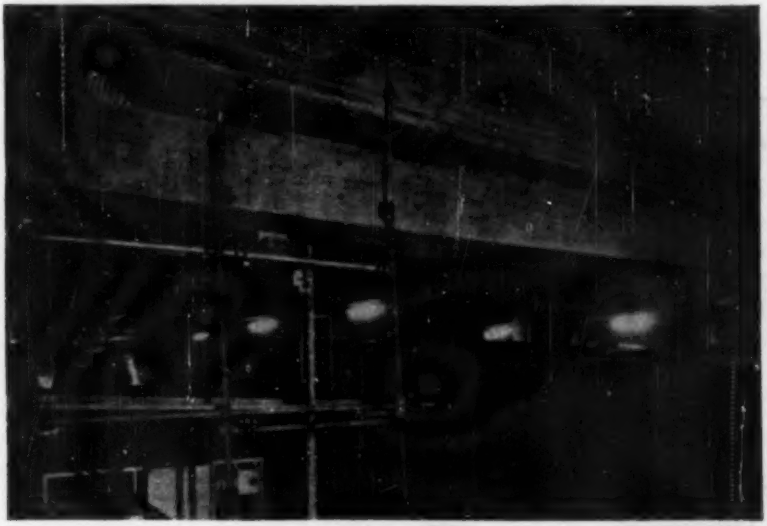

Fig. 1.

The strength of the concrete was specified to be 6000 lb. per square inch at 28 days. The cables were tensioned ten days after the beam had been concreted, and a week later were again tensioned to ascertain the reduction of prestress. The greatest reduction was 2 tons in one of the cables tensioned to 93 tons and was due almost entirely to the order in which the cables were tensioned, since a reduction of the force occurs in the cables tensioned first due to the elastic shortening of the beam caused by tensioning subsequent cables.

After grouting the cable ducts, gauges were placed between the beam to measure the deflection when the cast-iron column was removed. The column was cut first by means of oxy-acetylene flame at a convenient position near the floor. During the operation of burning through the column the head of the column and the beam rose 0·04 in. due to the thermal expansion of the column. Immediately after the column was cut the downward deflection of the beam was 0·01 in. but during a period of two days the deflection reversed to an upward movement of 0·01 in. The probable explanation of these movements is that the initial downward deflection was caused by the elastic shortening and settlement of the columns and that the reversal of the deflection was due to creep of the prestressed beam caused by an excess of bending moment due to the prestressing force exceeding the bending moment due to the load. The deflections were very small and measurable only with sensitive deflectometers, and the final upward camber was advantageous.

At the moment the column was cut there was a slight jerk in the structure, causing a sideways movement of about ¼ in. at the upper part of the column. This action indicated that the column was carrying the load eccentrically, thus producing bending moments the strains due to which were released on cutting the column. The column was later cut at a position 1 in. inside the beam by drilling and sawing at an angle; about a third of the holes had been drilled prior to casting the beam. The flat-jack on the new column did not have to be used owing to the satisfactory final deflections. Should there be any future differential settlement between the new and the old buildings it will be a simple matter to use the jack to counteract this effect.

The architect for the building is Mr. H. F. Bailey. G.K.N. Reinforcements, Ltd., made the structural design and supplied the reinforcement. The main contractors were Messrs. Holland & Hannen and Cubitts, Ltd.

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